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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

STABILITY OF STRAIGHT CONCRETE GRAVITY DAMS

BY D. C. HENNY,[1] M. AM. SOC. C. E.

SYNOPSIS

The necessity for abandoning the middle-third theory and the sliding factor as useful elements in masonry dam design is demonstrated in this paper. The writer develops the fact that the safety of straight masonry dams is principally due to shear, and he investigates the effect of loading on shearing strength. A résumé is given of available data on uplift pressure and estimates of effective uplift area and of uplift force are presented. Stability of dams of ideal triangular cross-section is studied and a shear safety factor is proposed for use in dam design.

The Levy requirement is found to be excessive for dams less than 500 ft in height under most adverse conditions of uplift, and more logical requirements fitting these conditions are suggested. Numerical examples are given of shear safety factors on the basis of cross-sections of existing dams. It is shown that the principles developed for straight concrete dams can be made equally applicable to other types of masonry dams.

HISTORICAL REVIEW

The middle-third method of finding safe dimensions of a gravity dam was commonly used until the beginning of the Twentieth Century. It merely contemplated absence of tension in the body of the masonry near the down-stream face with reservoir empty and near the up-stream face with reservoir full.

Compliance with this requirement was deemed to afford a factor of safety of two against overturning, assuming loading stresses to be moderate. When it became evident that failure of solid gravity dams was generally accompanied by down-stream mass movement and that this was facilitated if not in-

NOTE.—Discussion on this paper will close in December, 1933, *Proceedings*.

[1] Cons. Engr., Portland, Ore.

duced principally by penetration of pressure water at the base of the dam, or in the foundation, two new factors were taken into account, namely, uplift and sliding.

As early as 1882, danger from uplift was recognized and was considered in the design of the Vyrnwy Dam[2] of the Liverpool (England) Water System built in the period from 1882 to 1890. This dam was given a complete system of rock drains for the special purpose of reducing uplift.

The failure of the Austin (Tex.) Dam, in 1900, called forcible attention to the danger of sliding on foundation strata. It was realized that percolating water reduces friction, particularly when it is under pressure.

The first American dam in the design of which allowance was made for uplift is the Wachusetts Dam, in Massachusetts (1900–06). However, no drainage system was installed and the location of the up-stream cut-off in this as well as in the New Croton Dam, in New York State, is far down stream from the heel.

The Olive Bridge Dam, in New York State (1908–14), like the earlier Tytam and Cataract Dams, has drains in the masonry, but lacks foundation drainage.

Large dams in the United States, first provided with foundation and masonry drains to reduce uplift, are believed to be the Medina Dam, in Texas[3] (1911–12), and the Arrowrock and Elephant Butte Dams of the United States Bureau of Reclamation, in Idaho and New Mexico, respectively (1914–15). Foundations were drained to a considerable depth by drilled holes. Nearly all the high masonry gravity dams built in the United States since 1915 are provided with foundation and masonry drains.

The next step in dam design was due to experience which showed that drainage could not be depended upon to relieve, completely, the uplift under and in the body of a masonry dam. Allowance was made, therefore, in the design for both drainage and uplift and the following equation was developed:

$$F_s = \frac{P}{W - U} \qquad (1)$$

in which, for a unit length of dam, F_s is the sliding factor; P, the horizontal water pressure; W, the weight of masonry above an assumed sliding plane; and U, the uplift force under the sliding base. It was generally considered satisfactory if F_s was well below the coefficient of friction of masonry on itself, or on rock, the value of which was taken at a figure close to 0.75 in the case of hard rock foundation.

A further step is proposed in this paper, namely, the consideration of shear resistance rather than sliding resistance in ordinary cases. The basic formula proposed is:

$$Q = \frac{S}{P} \qquad (2)$$

in which, Q is the factor of safety in shear; S, the total shearing strength

[2] "Design and Construction of Masonry Dams," by Edward Wegmann, M. Am. Soc. C. E., Seventh Edition, p. 82.

[3] *Engineering Record*, December, 1912, p. 632.

that must be overcome before down-stream movement of any part or the whole of a dam can take place; and P, the water pressure on the projected area of the part assumed to move, on a vertical plane normal to the direction of motion.

Shearing Strength

Shearing strength is difficult to determine because in most shear tests there is interference from bending and resultant tension. There is likewise uncertainty regarding the distribution of shear. In addition, the important question arises as to whether shearing strength along any plane of movement is affected by the loading normal to that plane. The internal friction theory, originated by Coulomb, assumes that resistance to movement consists of two parts: (*a*) The pure shear strength encountered when there is no load on the plane of shear; and (*b*) an addition in the nature of friction, as expressed by:

$$s = s_1 + kq \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

in which, s is the total unit shearing strength; s_1, the strength under conditions of no load; q, the unit load on the plane of shear; and k, the factor of shearing strength increase (assumed by Coulomb to be constant).

This theory has not been proved. The conception, however, of an increase in shearing strength due to compression has been accepted by many engineers, partly on the basis of experiments and partly on theoretical reasoning. It is argued that compression lessens the distance between molecules, thereby increasing their mutual attraction and making relative movement more difficult.

It has also been reasoned in connection with the classical cubical unit element that shear induces diagonal tension which, in brittle material, may be the main cause of shear failure, and that this tension is diminished by mass compression.

Punching shear tests made by A. N. Talbot, Past-President and Hon. M. Am. Soc. C. E., in 1906,[4] gave results with concrete of 1:3:6 proportions, about 60 days old, as follows:

Slabs	Shearing strength, in pounds per square inch
A	936
B	1 025
C	1 688

The specimens were 13 by 13-in. concrete slabs, 3 in., 5 in., and 5 in. thick, respectively. Slab *B* was a recessed slab and thirty-two of these were punched. Slab *C* was of reinforced concrete.

The tests with Slabs *A* and *B* caused the formation of four radial cracks in the concrete surrounding the punch hole, and fine cracks apeared in the reinforced slabs, *C*. The formation of these cracks proves that tension was set up in the outer portion of the slabs and that inward pressure was exerted radially against the outer cylindrical face of the punched-out disk. With

[4] "Tests of Concrete," *Bulletin No. 1*, Eng. Exp. Station, Univ. of Illinois, Urbana, Ill.

the 5-in. recessed slabs (*B*), this pressure must have been greater than with those 3 in. thick (*A*), and it must have been still greater with the reinforced slabs (*C*). Correspondingly, with progressively greater pressure on the disk, higher values of shearing strength were found.

Great differences were found between the compressive strength observed on the tests of 6-in. cubes (2 075 lb per sq in.) and that apparently resulting from the tests of 8 by 16-in. cylinders (1 241 lb per sq in.). This is due, as is well known, to the fact that the breaking of a cylinder of the usual proportions is not basically a compression or crushing test; in reality, it is a

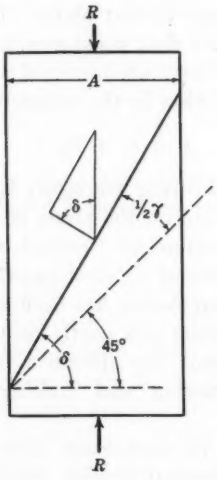

FIG. 1

structural test, measuring the strength of a concrete column, failure being due usually to diagonal shear. For this reason that class of test should yield information regarding shearing strength.

Fig. 1 illustrates a plane of breakage in a cylinder test. Assuming the correctness of the Coulomb theory, it can be shown[5] that, in Equation (3), with $\frac{R}{A} = r$:

$$s_1 = \frac{r}{2 \tan \delta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

and

$$k = \tan \gamma \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Since $\gamma = 2\ (\delta - 45°)$, the measurement of r and δ permit the determination of s_1 from Equation (3), provided the break occurred along a reasonably even shearing plane as in Fig. 1, or the more common cone break.

The angle of break in concrete is usually between the limits of 60 and 70 degrees. Values of k and s_1 are shown in Table 1.

[5] "Strength of Materials," by the late George F. Swain, Past-President and Hon. M. Am. Soc. C. E., First Edition, pp. 118–119.

TABLE 1.— VALUES OF k AND s_1

δ	γ	k	$\frac{s_1}{r}$	s_1 for $r = 3\,000$ lb per sq in.
60°	30°	0.58	0.289	867
62° 30′	35°	0.70	0.261	783
65°	40°	0.84	0.234	702
67° 30′	45°	1.00	0.207	621
70°	50°	1.19	0.182	546

Thus, if the angle, δ, of a 1:3:6 cylinder, 60 days old, measures 62° 30′ with $r = 3\,000$ lb per sq in., s_1 will be 783 lb per sq in., and $k = 0.70$; from which $s = 783 + 0.70q$.

TABLE 2.—TESTS OF CONCRETE CYLINDERS, UNITED STATES BUREAU OF RECLAMATION

Number of specimens	Size of cylinders, in inches	Concrete mixture	Maximum size of aggregate, in inches	Wet screened to (inches):	Age, in days	Break		$k = \tan \gamma$	r, in pounds per square inch	s_1, in pounds per square inch
						Kind	δ			
3	6 by 12	1:9.5	..	1½	28	Cone	73° 42′	1.56	4 300	670
3	6 by 12	1:9.5	..	1½	28	Diagram	60° 30′	0.60	4 260	1 200
3	8 by 16	1:9.5	..	3	28	Cone	68° 18′	1.06	3 540	700
6	8 by 16	1:9.5	..	3	28	Diagram	63° 42′	0.76	3 540	870
2	8 by 16	1:9.5	..	3	90	Cone	66° 30′	0.93	4 470	970
1	8 by 16	1:9.5	..	3	90	Diagram	67°	0.97	3 880	820
2	12 by 24	1:7.63	3	..	28	Cone	67°	0.97	3 490	740
1	24 by 36	1:8.92	6	..	28	Cone	68°	1.03	3 020	610
3	36 by 72	1:9.5	9	..	28	Cone	63° 18′	0.74	3 050	770
1	36 by 72	1:9.5	9	..	90	Diagram	67°	0.97	3 810	810

In 1932 some breakage tests were made in the laboratory of the U. S. Reclamation Bureau, at Denver, Colo., with cylinders of various sizes, ages, proportions, and maximum aggregates. The results are shown in Table 2.

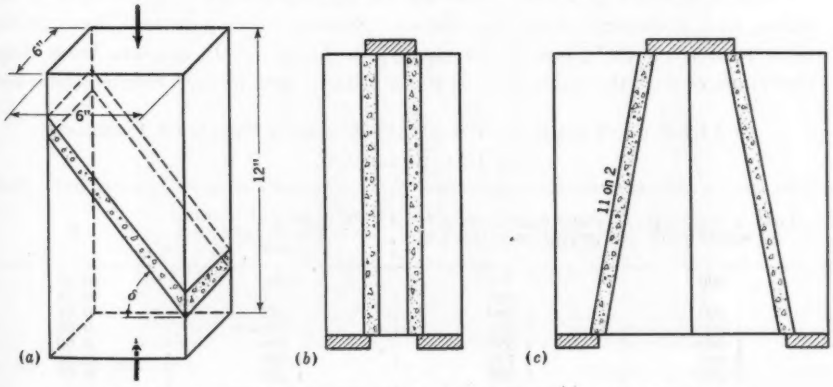

FIG. 2.— SPECIMENS TESTED BY S. H. WOODARD, M. AM. SOC. C. E.

It will be noted that there is considerable variation in the values found, as might be expected from the relatively small number of specimens tested and especially from the uncertainty of the effect of restraint at the ends.

Interesting experiments were made by Silas H. Woodard, M. Am. Soc. C. E., in 1925, with mortar specimens (see Fig. 2(*a*)), which were broken

at 28 days under vertical loading. They were made of 1:1 cement mortar, except that a ¾-in. zone of 1:3 mortar passed through each specimen at different angles with the horizontal. The object was to insure failure under load by shear in the weak zone and thus to ascertain the relation between shearing strength and loading on the plane of shear. In all cases breaks occurred in the weak zone. The component, $R \sin \delta$, of the breaking load, R (δ being the angle of the zone with the horizontal), was taken as the shear load, and $R \cos \delta$ as the loading normal to the plane of shear, in studying the results of Mr. Woodard's findings. The breaking loads, R, for the various values of tan δ, are given in Columns (2), (5), and (8) of Table 3, which, in Columns (3), (6), and (9), gives the shearing strength, s, for normal loading, q, both in pounds per square inch, in the manner explained previously.

TABLE 3.—Shearing Strength, s, Under Different Normal Loads, q.

Tan δ	First Series			Second Series			Third Series		
(1)	R (2)	s (3)	q (4)	R (5)	s (6)	q (7)	R (8)	s (9)	q (10)
0.40	154 760	1 480†	3 710†	145 650	1 390	3 480	156 270	1 500†	3 740†
0.80	141 170	1 920	2 400	138 850	1 880	2 340	141 070	1 920	2 300
1.20	98 320	1 345	1 102	115 170	1 613	1 301	95 210	1 330	1 076
1.75	90 300	1 070	615	86 340	1 030	590	99 000	1 180	680
2.33	81 880	954	414	99 700	1 180	502	91 350	1 180	460
∞	98 190*	682	0	100 640*	706	0	101 200*	690	0
—5.50	78 480	527	—96	57 650	384	—70	66 670	444	—81

* Double actual load, specimen breaking in one zone only.
† Specimens did not break.

The values of s in Table 3 were plotted against values of q, for each of the series, and a smooth curve was drawn through these points. Then, points were read from the curve and recorded in Table 4. It appears from these experiments that the value of k is not constant, and in that respect these test

TABLE 4.—Values of Factor, k, Shearing Strength Increase Due to Load, q.

Load, q, in pounds per square inch	Shearing strength, s, in pounds per square inch	Shearing strength increase, kq, in pounds per square inch	k
—100	400	—290	0.35
0	690	0	0.74
500	1 060	370	0.74
1 000	1 350	660	0.66
1 500	1 580	890	0.60
2 000	1 770	1 080	0.54
2 500	1 940	1 250	0.50

results do not agree with the theory of Coulomb. The important fact is evident however, that for 1:3 mortar there is a persistent increase of shearing strength with increasing load; that within limits of loading from 0 to 1 000 lb per sq in., k is fairly constant and may be taken at 0.70; and that the shearing strength of the mortar tested may be computed as $690 + 0.70\ q$.

The strength of 28-day concrete test cylinders (4 by 8 in.) has been measured at the University of Illinois,[6] under conditions of axial and tri-axial loading. The mix was 1 : 2.1 : 2.5. The breaking load was applied axially and the circumferential load was produced by oil pressures ranging from 180 to 4 090 lb per sq in. This pressure appeared generally to prevent the occurrence of a shear break, but a clean break resulted in one case (Specimen 114). The breaking load was 2 840 lb per sq in., the oil pressure, 180 lb per sq in., and the shear angle, 65° 17′. From this one experiment, a value of $s = 649 + 0.664q$, would result. Single experiments of this type can have but little weight; yet the one quoted herein conforms satisfactorily to the theorem that shearing strength increases with load.

Near the base and along the abutments of a dam the shearing strength of the foundation rock is involved, in regard to which little is usually known. Tests with granite, limestone, and sandstone in building blocks have shown strengths ranging from 1 000 to 2 800 lb per sq in.

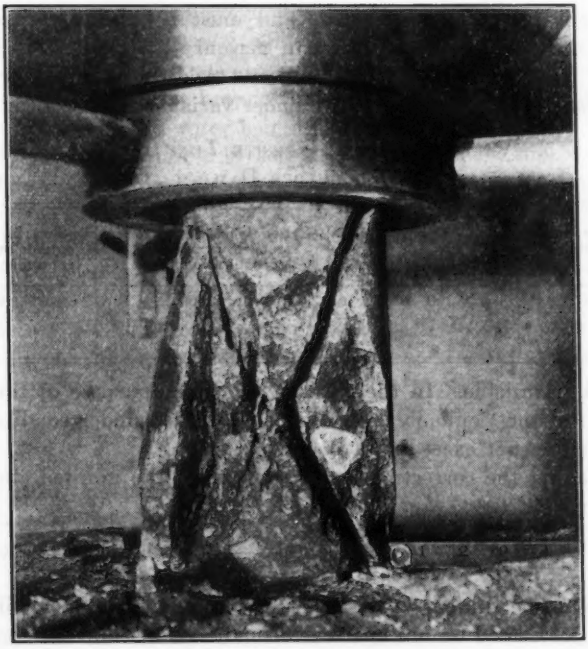

FIG. 3.—COMPRESSION TEST OF DRILL CORE; SPECIMEN OF ANDESITE TUFF BRECCIA FROM ABUTMENT OF BOULDER DAM.

Compression tests with cylinder specimens of rock may give some information. Breaking tests of drill cores taken from the rock at the abutments of Boulder Dam may be cited. The cylinders were 4.75 in. in diameter and were cut 9.5 in. long. Fig. 3 shows a broken sample in the testing machine

[6] *Bulletin No. 185*, Eng. Exp. Station, Univ. of Illinois, pp. 56, 57, and 59.

and Fig. 4 shows parts of eight other samples. The angle of the break in Fig. 3 is 66° and, in Fig. 4, it varies from 65 to 70 degrees. The minimum load at failure for a thoroughly soaked specimen was 11 700 lb per sq in.,

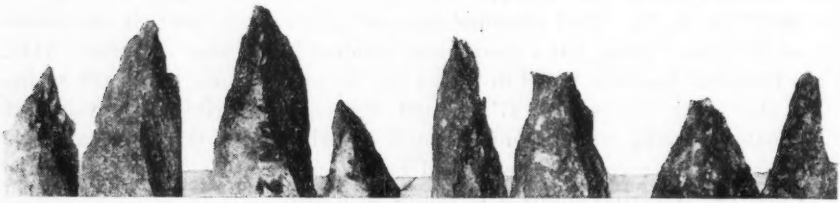

FIG. 4.—TESTS OF ROCK SAMPLES (ANDESITE, TUFF-BRECCIA), BORED FROM ABUTMENT FOUNDATION, BOULDER DAM.

and the average was 17 000 lb per sq in. Using the lower value (11 700 lb per sq in.), Table 5 was prepared on the basis of Equations (4) and (5).

The experiments herein quoted are by no means conclusive as to safe values of shear. It is remarkable (and must be seriously regretted, since shear is a common cause of failure in masonry), that there is such a great paucity of published data on this subject. Additional laboratory tests have been suggested, such as torsion tests under variable axial loading, and results

TABLE 5.—VALUES OF s_1 IN ANDESITE TUFF BRECCIA, ROCK CORES, BOULDER DAM

Angle of shear break with horizontal in degrees, δ	Factor of shear increase, k	Cylinder test; compressive strength, in pounds per square inch, r	Shearing strength, in pounds per square inch, under no load, s_1
65	0.84	11 700	2 740
66	0.90	11 700	2 610
70	1 20	11 700	2 140

may soon be available. In the meantime, for the purpose of this paper, the broad truth of the Coulomb theory will be assumed under conditions of compression loading not exceeding 1 000 lb per sq in.

Thus, taking the correctness of Equation (3) for granted and combining it with Equation (2):

$$Q = \frac{s_1 + kq}{P} \text{ (base area)} = \frac{S_1}{P} + k\,\frac{W - U}{P} \quad \ldots\ldots\ldots\ldots (6)$$

in which, S_1 is the full base shearing strength under no load condition. Equation (6) is predicated on unimpaired intimate contact and adherence over the entire base.

UPLIFT PRESSURE

In ordinary cases of good rock foundations the pressures at the base of the dam decrease in the down-stream direction so that the pressure gradient falls well below a straight line that extends from full reservoir head at the up-stream face to zero at the toe. This has been shown[7] convincingly by

[7] *Civil Engineering*, Vol. 2, No. 9, September, 1932, p. 578.

Ivan E. Houk, M. Am. Soc. C. E. It was also found that the pressure at concrete construction joints was quite small, and, in some cases, could not be measured.

The average uplift pressure under existing dams has been reported[8] as follows:

Dams.	Average pressure, in feet, in terms of reservoir head, H	
	Drained	Undrained
Neye, Germany	.....	0.349 H
Brule River, Wisconsin	0.336 H	.....
Medina, Texas	0.290 H	.....
Gibson, Montana	0.248 H	.....
Willwood, Wyoming	.....	0.246 H
Pit River, California	0.190 H	.....
American Falls, Idaho	0.153 H	.....
Ariel, Washington	0.121 H	.....
Bull Run, Oregon	0.097 H	.....

The direct effect of drainage on uplift pressure is difficult to determine. At times, it is very material as was shown by the test closing of a rock drain at Ariel Dam, which caused the pressure to rise at a point 16 ft from the water face from 0 to 80% of the water height.

Drains are likely to close and become clogged. They should be straight and accessible for periodical cleaning, but there is no certainty that this will not be neglected. Conservatism in design dictates that no dependence should be placed on the perpetual reduction of uplift pressure by drains. It is proposed, therefore, in estimating uplift pressure to use an even gradient from H at the water face to 0 at the down-stream face. This is equivalent to an average pressure along any horizontal plane of $0.50H$, which is seen to be from 1.5 to 5 times the pressure measured in drained dams and from 1.5 to 2 times that found in dams that are not drained.

Effective Uplift Area

The uplift force, with which the designer of dams is principally concerned, is the product of pressure and area. The latter factor may be of even greater importance than the former, but the determination of the area over which uplift pressure is effective is far more difficult than the measurement of pressure.

As regards pore-water uplift a fairly accurate estimate of area is possible. The pore space and, consequently, the pore area, in good concrete cut by any plane will probably occupy from 10 to 12% of space and area, respectively.

Effective uplift area—in joints, in incipient cracks, along the junction of rock and concrete, or in seams in the rock just below the base—is a factor that has perplexed designers because of the complete absence of known facts and because of the variability of local conditions.

In an effort to throw some light on this subject as regards concrete construction joints, tests were conducted at the laboratory of the Bull Run Dam

[8] *Civil Engineering*, Vol. 2, No. 9, September, 1932, p. 580.

in 1927. The results were negative in that they indicated that bond between concrete and concrete, as made in the laboratory (even in careless work) is practically as tight as the concrete above and below the bonded plane. Similar results were obtained[9] two years later in Pasedena, Calif., by the late John Egon Skafte, Jun. Am. Soc. C. E., under the supervision of Franklin Thomas, M. Am. Soc. C. E.

The fact that occasionally water under pressure has been found in field construction joints and that seepage water has been frequently observed to issue at the intersection of such joints with the down-stream face justifies the conclusion that water movement may be due to imperfections in mass concrete regarding which laboratory work can give no information other than that, with care, a tight bond can be secured.

Nothing is known of effective uplift area at the base of a dam where concrete comes in contact with rock. All that research has revealed thus far is that, at the great majority of points at which measurements were attempted, both pressure and measurable flow were found. From this it may be inferred that the relative effective area is greater at the base than in concrete construction joints. This conclusion seems reasonable when the physical differences are considered between the two materials in contact. These refer in particular to the elastic moduli and the coefficients of expansion by heat, both of which are certain to set up severe local stresses which may result in separation. Furthermore, the foundation may contain deep-seated seams that may admit water to the base around any cut-off or grouting screen that is provided. It has not yet been possible to appraise these factors which make the problem a practical rather than a theoretical one.

There is one factor, however, that must have considerable influence on potential uplift area, namely, intensity of loading. It is evident that heavy compression at contact faces must greatly hinder, or even stop, all percolation except through pore areas. Inversely, in the case of light loading, the uplift force in areas of imperfect bond and in pores may put the concrete in tension.

If H is the depth, in feet, below water of a point on the water face, and therefore, likewise expresses the maximum intensity of uplift pressure per square foot (62.5-lb unit) near the water face, and if e is the proportion of uplift pressure area to gross area around that point, then eH is the unit uplift force at that point.

It is conceivable that with no loading and with perfect adhesion, separation would not necessarily follow. However, dependence cannot be placed on tensile strength at concrete construction joints and especially at bond between concrete and foundation rock. When eH is greater than unit loading, tension will exist. Should a tension break occur, the uplift pressure area will increase and the resultant uplift force, acting as a wedge, will tend to cause progressive separation and to an increase of pressure area. It is of great importance that this be avoided, and to that end surplus vertical loading must be provided at the water face.

[9] Unpublished thesis, California Inst. of Technology, Pasadena, Calif., 1929.

Many small dams have been built with a vertical water face and a downstream slope of 0.70, which corresponds to a heel loading (reservoir full), of about 0.36 H. As, in general, no progressive increase of percolation or uplift pressure has been noted it may be argued that the effective uplift area at the water face did not exceed 36 or, say, 40 per cent. This percentage equals more than three times the pore area and is suggested as a fair and safe estimate under such loading conditions, taking careful field work for granted.

For lighter loading it may be supposed that this percentage increases rapidly. On the other hand, with heavier loading, the potential percentage will probably decrease and the somewhat arbitrary relation listed in Table 6 (Columns (2) to (5)) may be reasonably considered, on the assumption that the dam is founded on good rock.

TABLE 6.—ASSUMPTIONS AS TO UPLIFT

Vertical loading at the heel	Effective Uplift Pressure Area (Percentage of Total)				Average uplift pressure, in 62.5-lb. units	Uplift force, U, in 62.5-lb. units
	At heel	At toe	Average	Relative area subject to uplift (proposed for design)		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0.4 H	40†	25	32.5	40	0.5 H	0.200 HL
0.6 H	35	25	30.0	35	0.5 H	0.175 HL
0.8 H	30	25	27.5	30	0.5 H	0.150 HL
1.0 H*	25‡	25	25.0	25	0.5 H	0.125 HL

*Allowance for pore-water pressure. † Three times pore area. ‡ Twice the pore area.

It would be logical when considering the entire base to use the percentages in Column (4) of Table 6. It must be remembered, however, that it is mostly imperfections in concrete and rock base that govern these percentages and that an accurate appraisal of these imperfections is practically impossible. The more conservative course is recommended, therefore, namely, to use the values in Column (5) for the entire base.

UPLIFT FORCE

The estimates for design made first of uplift pressure and next of uplift area may be combined in order to arrive at the uplift force (see Table 6, Columns (6) and (7)). In Column (7), Table 6, L is the length of base, in feet.

The existence of an uplift force due to water under pressure in the pores of concrete has been questioned. This argument has been definitely settled, however, at least in the case of mortar, by unpublished experiments made by Mr. Woodard. Mr. Woodard tested the middle part (2 in. high), of the cylindrical surface of 2 by 4-in. mortar specimens of various mixtures and subjected them to water pressures up to 2 000 lb per sq in. in an apparatus that permitted the ends to be left free and exposed to the air. The water entering the pores percolated slowly toward the outer ends, and in the interior it exerted pressure which, acting in the pore spaces, produced an axial force sufficient to break the specimen.

The breaking force was estimated from the hydraulic pressure necessary to cause the break and the ascertained porosity. The tensile strength of the specimens was found by breaking control briquettes. Column (7), Table 7, shows the relation between the force necessary to break the specimen and the force computed to be required from the tensile strength.

That the results are far from consistent may be partly explained by the fact that the briquettes may not have furnished correct values of the tensile strength of the cylinders. Furthermore, the water pressure in the pores of

TABLE 7.—WOODARD TESTS TO DETERMINE FORCE OF WATER IN THE PORES OF MORTAR

Number of specimens broken	Mixture	Average Value				
		Hydraulic breaking pressure, in pounds per square inch	Porosity (percentages)	Breaking force, in pounds per square inch	Strength of briquettes, in pounds per square inch	Breaking force / Strength of briquettes
(1)	(2)	(3)	(4)	(5)	(6)	(7)
18.....	Neat	1 411	31.0	438	477	0.92
11.....	1:1	1 485	25.9	384	454	0.85
24.....	1:2	1 514	20.3	305	475	0.64
22.....	1:3	959	17.1	164	437	0.37
10.....	1:4	191	19.3	37	349	0.11
10.....	1:5	165	25.3	42	240	0.17
12.....	1:6	122	29.5	36	173	0.21

any normal section cannot have been uniform throughout the full area; it was undoubtedly smaller at the center than at the circumference. The average may have easily been a different proportion of the measured outer pressure for the various mixtures. The important fact has been demonstrated, however, that water in pores can exert an uplift force.

Accepting, as a fact, the possible existence of uplift force due to the pressure of water in the pores of mortar, there is every reason to conclude that a similar force may be exerted in the body of concrete even if its potential amount will probably be less because of smaller pore space.

The average pressure in pores may be considered as governed by the same law as that in fine seams. Therefore, a value of $0.5H$ was assumed in Columns (6) and (7), Table 6. This table, in Column (7), contains recommended safe estimates of uplift force in dams in which the vertical loading at the up-stream face varies from H to $0.4H$. At this lower limit of loading (reservoir full) a balance may be estimated to exist between local uplift force and vertical loading at the water face. In dams with still lighter loading this may be exceeded by uplift force, and tension may develop in the concrete. As was argued under the heading "Effective Uplift Area," the danger of this condition lies in a possible increase of uplift area, resulting, of course, in proportionate force increase.

EFFECT OF UPLIFT FORCE ON STABILITY

In studying principles governing the effect of uplift on stability it is convenient to consider the unit of an ideal dam of infinite length with tri-

angular profile having a vertical water face, a horizontal base, and water at the apex level, as shown in Fig. 5(*a*).

Water pressure in the pores of concrete implies permeability. Concrete must be workable when it is placed and usually more water is required for this purpose than is necessary for completing chemical action. Therefore, the interior of massive concrete may be expected to have its pores filled with water if air has been excluded in placing and if no loss has been suffered from surface evaporation.

Excavation and core drilling in dam concrete have shown the mass to remain fully saturated for an indefinite time even without water against

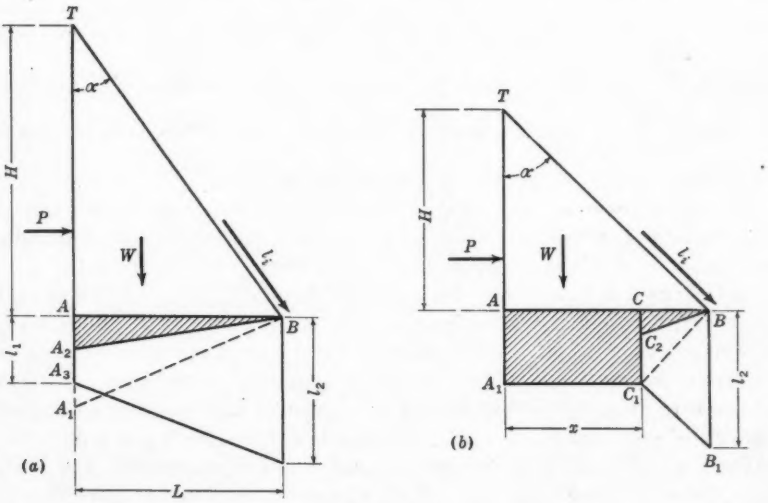

FIG. 5.

the dam except that an outer shell will have partly dried. The depth of this shell is relatively small in a large dam.

Concrete having 12% pore space, and with an average specific weight of aggregates varying from 2.65 to 2.60, will have a mass specific weight, oven dry, ranging from 2.34 to 2.29, and when fully saturated from 2.49 to 2.44. The weight of pore water above any sliding plane and the pore-area uplift on such plane will tend to balance each other. Since pore uplift will be considered separately the specific weight of mass concrete must be computed on the basis of concrete that is mostly saturated and will be taken at 2.4 in this study.

The potential effect of heel loading on uplift pressure area and force has been discussed. In order to estimate uplift force, the toe loading must be known. In this paper it is proposed to determine the toe loading on the assumption of straight-line distribution of normal foundation stresses. This assumption does not accord with the elastic theory, but is made because the great uncertainties as to uplift forces do not justify great refinement

in a general treatment of the subject. It should be understood, however, that in important individual cases such refinement may be desirable and important.

It can be shown that, on the assumption of a linear distribution of normal stresses and no effect on loading of uplift (see Fig. 5(*a*)),

$$l_1 = H\left(a - \frac{1}{n^2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

$$l_2 = \frac{H}{n^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

and,

$$l_t = H + \frac{H}{n^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which, H = height of dam, in feet; L = width of base of dam, in feet; $n = \tan\alpha = \frac{L}{H}$; α = angle between the up-stream and the down-stream faces of the dam; and, a = specific weight of concrete.

The uplift pressure may be represented, as previously stated, by a straight line, such as $A_1 B$ in Fig. 5(*a*). The uplift force may be correspondingly represented by the line, $A_2 B$, if AA_2 bears a proper relation to AA_1.

So long as AA_2 is less than AA_3 (the heel loading), no alteration of loading occurs. If, however, the summit angle is so small that l_1 is less than the limit previously estimated at $0.40H$, there is danger that uplift force will cause progressive separation. This will be accompanied by increase of pressure and pressure area, which may extend to a point of the base where the intensified vertical loading, due to load shift, equals full reservoir pressure.

Thus, a condition may be created, such as is illustrated in Fig 5(*b*), in which the entire area, $AA_1 C_1 B_1 B A$, represents the weight of the dam; and AC is the length of dam base over which gradual separation due to uplift has taken place, and for which the uplift pressure may be equal to the full water height effective over 100% of the area. The loading equals full water pressure, H, at Point C, and the triangle, CC_2B, represents pore-area uplift force.

For such a case (see Fig. 5(*b*)) assuming the specific weight of concrete to be 2.4:

$$x = \frac{2.5 - 3.5\,n^2}{n} H \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$l_2 = \frac{4.9\,n^2 - 2.5}{4.5\,n^2 - 2.5} H \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

and,

$$l_t = \frac{(4.9\,n^2 - 2.5)\,(1 + n^2)}{4.5\,n^2 - 2.5} H \quad \ldots\ldots\ldots\ldots\ldots (12)$$

From Equation (10) it will be seen that $x = 0$ for $2.5 - 3.5n^2 = 0$, from which, $n = 0.843$, which is the condition proposed by Levy; that is, loading at the water face must not be less than the water pressure at that elevation.

It will also be noted from Equation (11) that the vertical toe loading, l_2, will become ∞ for $4.5n^2 = 2.5$, or $n = 0.743$.

Thus, it would appear that when the rock foundation is so bad that unusually great uplift pressure should be expected, a value of n greater than 0.743 must be used to prevent crushing at the toe. This would not mean, however, that a value of n of 0.843 would be necessary under all conditions, as advocated by Levy. Even with a lower value of n there may yet be ample resistance along the portion, BC, of the base to give the required security against down-stream movement.

Under ordinary conditions of reasonably tight foundation and on the basis of assumptions previously made, the relation in the ideal dam of dimensions and uplift force is shown in Table 8, Items Nos. 1 to 6. In order to evaluate

TABLE 8.—RELATION BETWEEN DIMENSIONS OF AN IDEAL DAM, UPLIFT FORCE, SHEAR SAFETY FACTORS, AND NOMINAL SLIDING FACTORS

Item No. (1)	Description (2)	Heel Loading, Reservoir Full						
		$0.4H$ (3)	$0.5H$ (4)	$0.6H$ (5)	$0.7H$ (6)	$0.8H$ (7)	$0.9H$ (8)	$1.0H$ (9)
1	Ratio, $\frac{L}{H} = n$	0.707	0.725	0.745	0.766	0.790	0.815	0.843
2	Percentage of base area subject to uplift pressure	40.0	37.5	35.0	32.5	30.0	27.5	25.0
3	Average uplift pressure	$0.5H$	$0.5H$	$0.5H$	$0.5H$	$0.5H$	$0.5H$	$0.5H$
4	Uplift force, U	$0.141H^2$	$0.136H^2$	$0.130H^2$	$0.124H^2$	$0.118H^2$	$0.112H^2$	$0.106H^2$
5	Weight, W, of a linear foot section of dam	$0.848H^2$	$0.870H^2$	$0.894H^2$	$0.910H^2$	$0.948H^2$	$0.978H^2$	$1.013H^2$
6	Difference, $W - U$	$0.707H^2$	$0.734H^2$	$0.763H^2$	$0.785H^2$	$0.829H^2$	$0.865H^2$	$0.907H^2$
7	$1.4\frac{W-U}{H^2}$	0.99	1.03	1.07	1.11	1.16	1.21	1.27
8	$1.4\frac{W}{H^2}$	1.19	1.22	1.25	1.28	1.33	1.37	1.42
9	Increase in factor of safety, Q, if uplift is eliminated	0.20	0.19	0.18	0.17	0.17	0.16	0.15
	Factor of safety, Q, in Shear:							
10	$H = 200$	7.49	7.71	7.92	8.16	8.43	8.71	9.03
11	$H = 300$	5.32	5.48	5.64	5.81	6.00	6.21	6.44
12	$H = 400$	4.24	4.37	4.50	4.64	4.79	4.96	5.15
13	$H = 500$	3.59	3.71	3.81	3.93	4.07	4.21	4.37
14	$H = 600$	3.11	3.25	3.35	3.46	3.58	3.71	3.86
15	$H = 700$	2.85	2.93	3.01	3.11	3.23	3.35	3.49
16	Safety factor against nominal sliding, on the basis of a friction coefficient of 0.75	1.06	1.10	1.15	1.19	1.24	1.30	1.36

the probable effect of uplift on stability, Equation (6) may be used, in which it becomes necessary to assign values to s_1 (unit shearing strength under no load) and k (factor of shearing strength increase due to load).

Until the results of additional research become available the experiments herein quoted, so far as they go, justify a value of 0.70 for k and of 800 lb per sq in. for s_1. For sound rock the value of s_1 is much higher, but in mass formation it is subject to far greater uncertainty. In view, also, of uneven distribution of shear (the maximum being generally less than twice the average), and the possibility that some areas of construction joints in the dam, although staggered or stepped, may coincide with a potential plane of

shear movement—it is proposed in this study to give s_1 a value of 400 lb per sq in. In the units of previous equations this is equivalent to $400 \frac{144}{62.5} = 920$.

Introducing these values in Equation (6) and considering that the base area is nH and the horizontal water pressure, $P = \frac{1}{2}H^2$:

$$Q = \frac{1\,840\ n}{H} + 1.4 \frac{W - U}{H^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

In order to indicate the relative effect of uplift, the values of the second term of Equation (13) are shown in Items Nos. 7 and 8 of Table 8, in cases of uplift and no uplift.

Item No. 9 of Table 8 shows the numerical decrease in Q, due to uplift. Its relative importance can be judged by comparison with values of Q, found by Equation (13) and listed as Items Nos. 10 to 15, in Table 8. Comparing these items, involving uplift, with Item No. 9, it is found that in cases where Q equals 4 or more the factor of safety increase resulting from eliminating uplift, ranges from 3% to 5%, assuming that the water-face loading, with reservoir full, does not drop below $0.4H$, and that a reasonably tight foundation exists.

Where the foundation is so seamy and permeable as to permit conditions to arise, as illustrated in Fig. 5(*b*), a far more serious effect should be anticipated although, even then, safety may be obtained by increasing the dimensions.

The problem requires special treatment when the foundation contains extensive horizontal stratifications. In such a case, movement on or in the foundation may take place without overcoming shear resistance and may merely encounter frictional resistance, becoming thereby equivalent to the cases of loose rock-fill and earth dams. Then, uplift may have its most serious, and easily its most dangerous, effect on stability unless dam dimensions have been greatly increased. The mere presence of water in horizontal foundation seams tends to reduce the coefficient of friction; and, in addition, with water under pressure the friction-producing load may be greatly reduced. This was well illustrated in the failure of the dam at Austin, Pa.[10]

Caution in designing a dam will also be particularly important when the rock foundation is of a nature such as will cause the material to soften in the long-continued presence of water. After slow and almost unavoidable water percolation, especially where no deep cut-off is provided and base drainage is neglected, settlement must be expected, with dams of usual dimensions, at the down-stream toe. This settlement is likely to be followed quickly by freer access of water under the up-stream toe, which will result in nearly complete destruction of the shearing resistance of the base. At the same time, through uplift, it will result in a large reduction of frictional load and its resultant friction, which is all that remains to oppose the down-stream movement. It is believed that the failure of the St. Francis Dam started in that manner.

[10] *Engineering Record*, Vol. 7, 1911, p. 429.

Effect of Height on Base Length

Table 8 (Items Nos. 10 to 15) shows in the ordinary case of good rock foundation, the factor, Q, for dams of various heights and indicates that, comparing dams of the same proportions, the higher the dam, the less safe it is. This fact may be also directly inferred from Equation (13) in which the second factor, $1.4\,\frac{W-U}{H^2}$, is for dams of common section, practically, and for dams of triangular section, entirely independent of H, since W and H both vary with H^2, and H cancels out. The first factor, $\frac{1\,840\,n}{H}$, is decidedly the major item, and this factor becomes smaller with increase of height.

The economic profile of a dam with vertical face, considering only the element of down-stream mass movement, may be readily determined for any factor of safety assuming shearing strength to be known. If, as has been suggested, $0.4H$ is taken as the lowest permissible face loading, a value of $n = 0.707$ follows for the upper part of the dam where, with such value, shearing strength is excessive. Introducing this value in Equation (13):

$Q = \frac{1\,301}{H} + 1.4\,\frac{W-U}{H^2}$; and since $W = 1.2\,H^2 \times 0.707$ and $U = 0.2\,H^2 \times 0.707$; $Q = \frac{1\,301}{H} + 0.99$. For $Q = 4$, the head, H, is found to be 432 and the base length, L, becomes 305.4, all linear dimensions being in feet.

For heights greater than 432 ft, Q will become smaller than 4 unless the down-stream slope is flattened (see Fig. 6). By the further use of Equation

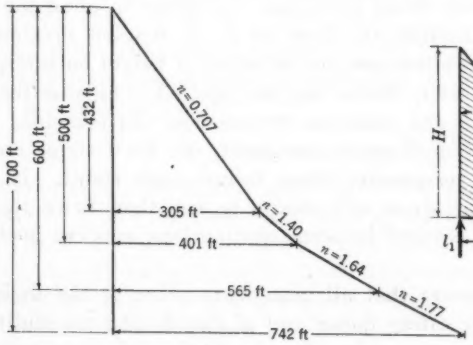

Fig. 6.

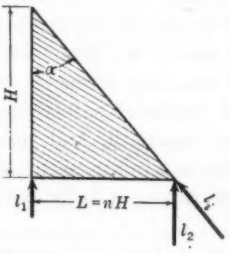

Fig. 7.

(13), it will be found that for $H = 500$, 600, and 700 ft, respectively, the required base lengths are 401, 565, and 742 ft, all on the following assumptions pertaining to a vertical-face profile (see Fig. 6): Specific weight, a, of concrete, 2.4; minimum loading on the water face, $0.4H$; uplift force, as given in Table 6, Columns (6) and (7); shearing strength, 400 + 0.709; and minimum factor of safety, Q, in shear, 4.

Safety limitation of the stresses in the concrete has been disregarded for the sake of simplicity, but it can be easily introduced into the problem. For instance, if 600 lb per sq in. is determined as the maximum compressive stress, this limit with reservoir empty would be reached, in a dam with a vertical up-stream face, at a height of 575 ft. Below this elevation it would be necessary to introduce a slope to the water face which, in turn, would cause a change in the down-stream slope.

The important fact to which attention is directed is that the maintenance of a definite safety factor in shear, for high dams, will require a rapidly increasing relative base length.

Shear Safety Factor *Versus* Sliding Factor

The general use that has been made in recent years of the sliding factor in judging the sufficiency of a dam profile, may have originated in the consideration that masonry, and especially concrete, dams are often constructed in horizontal courses between which adherence may be imperfect and such use may have been favored by the simplicity of the method and possibly also by the relative lack of knowledge of shearing strength.

The continued use of the sliding factor is incomprehensible nevertheless now that the necessity of securing a maximum of shear resistance by roughening and cleaning the base and by stepping or sloping the construction joints is so thoroughly understood and practiced, that simple sliding is out of the question.

The only logical use of the sliding factor may be found in case the aforementioned essentials are disregarded, or where the foundation rock is made up of extensive horizontal, or almost horizontal, strata.

In ordinary cases of a reasonably sound foundation free from such strata the use of the shear safety factor is logical. It brings to mind the necessity of designing and constructing the dam so as to develop maximum shear resistance and makes due allowance for the effect of height on safety.

The use of the shear safety factor may be objected to because the distribution of shearing strength and shearing stresses may not coincide. Thus, in spite of apparent safety as to mass movement, the local stress may exceed the local strength, and progressive shear failure may result. In ordinary cases the maximum shear stress will always be less than twice the average, and this range is fully covered by such assumptions as were made in this study.

On the whole, it appears that all arguments point to the logic in dam design of using the shear safety factor and of abandoning the sliding factor, except only where sliding may take place on horizontal construction joints or foundation strata without involving shear. Only in such case can the use of the sliding factor be considered logical and if so used, a suitable safety factor should be introduced.

Unusual Uplift Conditions

At some dam sites especially severe conditions may occur, such as possibilities of vibration due to heavy overflow or to earthquakes, or badly

seamed foundation, which may render the estimate of uplift pressure area in Table 6 unsafe. In such cases recourse may be had to studying the worst uplift conditions readily conceivable as illustrated in Fig. 5(*b*).

Shear value as well as frictional resistance for the base length, AC (see Fig. 5(*b*)) must be considered destroyed, all resistance to motion being confined to the remaining contact along the base length, CB. On the basis of unit values previously used, shear resistance is,

$$920\ (Hk - x) + 0.70\ (1.2Hn - U)$$

The value of uplift up stream from C is Hx, and down stream from C, where it results from pressure in pore area only, it is $0.125H\ (Hn - x)$, hence,

$$U = Hx + 0.125\ (Hn - x)H \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Introducing for x the value given in Equation (10):

$$U = 0.125\ H^2\ \frac{17.5 - 23.5\,n^2}{n} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

A factor of safety of four against movement is obtained by compliance with the equation:

$$4P = S + 0.7\ (W - U) \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The final safety requirement may be that the inclined toe loading (Fig. 7), shall not exceed a certain limit, say, 600 lb per sq in., which leads from Equation (12) to: $600 \times \frac{144}{62.5} = \frac{(4.9\,n^2 - 2.5)\ (1 + n^2)}{4.5\,n^2 - 2.5} H.$

The different values found from these equations are plotted in Fig. 8 as is also the Levy requirement which, by Equation (10), gives a value to n of 0.843. It will be noted that control is had for dams of less height than 575 ft by Curve Q (Fig. 8), which automatically provides for the inclined stress limitation. The spread between Curve Q and the line representing the straight-line Levy requirement is emphasizd in Fig. 8 by shading, and indicates its unnecessary excess.

Numerical Examples

It may be useful to present some numerical values of the sliding and the shear safety factors for cross-sections of a few existing dams. This is done in Table 9 on the basis of considering a linear foot of dam by itself and assuming that it can move down stream by overcoming base shear only. The same shearing strength values are used as heretofore, although careful investigation would undoubtedly have assigned different values to each individual case. Table 9 makes it additionally apparent that the shear safety factor furnishes a more rational criterion by which to judge the safety of dams than the sliding factor.

General Remarks

It should be emphasized that while the shearing strength of concrete may be estimated on a reasonable basis of experiment there remains a great uncertainty as to the mass shearing strength of foundation rock. It may vary

materially from place to place, and especially between river valley and abutment sections. This uncertainty will naturally lead to the adoption of a very conservative value, such as was used in this study. Even this may be far too high in the case of poor foundation rock.

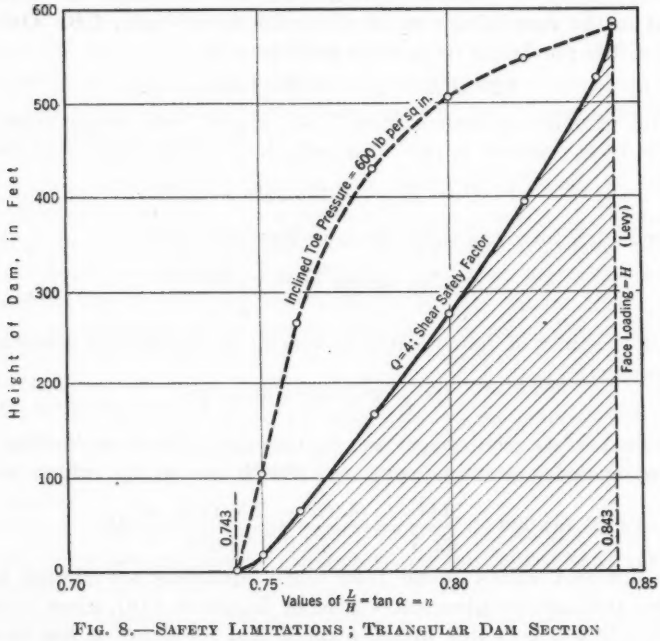

FIG. 8.—SAFETY LIMITATIONS; TRIANGULAR DAM SECTION

The shear safety factor may be determined easily for any linear foot of a dam if shear resistance on the base is the only item taken into account. However, allowance must be made for shear resistance which may develop

TABLE 9.—FACTORS OF SAFETY AGAINST NOMINAL SLIDING AND SHEAR

Description	American Falls	Bull Run (considered straight)	Elephant Butte
Height, in feet	79.5 below high water	200	254
Base width, in feet	60	148.5	205.6
Slope of water face	Vertical	25 on 1	16 on 1
Dry slope	3 on 2	7 on 10 and 8 on 10	2 on 3 and 1 on 1
Horizontal force *	3 052	19 950	30 600
Weight on foundation, W*	5 970	36 900	62 650
Heel loading *	0.44H	0.60H	0.69H
Ratio of effective uplift area to gross area	39%	35%	33%
Uplift force, U*	930	5 470	10 680
$W - U$*	5 040	31 430	51 970
0.70 ($W - U$)*	3 530	22 000	36 400
Base shear (no load), basis, 400 lb. per sq. in.*	55 200	136 400	189 100
Total base shear*	58 700	158 400	225 500
Ordinary sliding factor	0.61	0.63	0.59
Apparent factor of safety (basis, sliding factor, coefficient of friction, 0.75)	1.23	1.19	1.27
Shear safety factor, Q	19.1	7.9	7.3

* 62.5-lb. units.

at the ends of the section considered. The safety factor may also be determined for the dam as a whole not only in the instance of a straight gravity dam, but also for curved gravity, gravity arch, and arch dams. In such cases a trough-like surface may be assumed closely approximating concrete and rock contact along which shear movement would most likely take place. Such a general surface would have horizontal contours parallel to the direction of the supposed mass movement, or circular contours if rotary motion is considered about some vertical axis in or near the end of the dam.

Cantilever and arch stresses along the surface of such a trough will produce resultants, the aggregate horizontal components of which in the direction of motion, make up the total shearing force, which should equal the water pressure on a projected plane normal to the direction of motion. The aggregate numerical value of all components normal to the trough surface, less the corresponding uplift force, furnishes the load that causes an increase in shear, while the total area of the trough, less that part, over which adherence may be destroyed by uplift or tension from other causes, may be taken as the source of shear resistance exclusive of the increase due to load.

The rules of uplift determination would be the same as those suggested in connection with straight gravity dams, the principle being identical in all cases.

Acknowledgment

Acknowledgment is due to Silas H. Woodard, M. Am. Soc. C. E., for permission to use the valuable results of his research relating to mortar shearing strength and stresses resulting from pore water under pressure in mortar cylinders.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

ESTIMATING THE ECONOMIC VALUE OF PROPOSED HIGHWAY EXPENDITURES

BY THOMAS R. AGG,[1] M. Am. Soc. C. E.

Synopsis

This paper presents a discussion of a method whereby a proposed highway improvement can be analyzed to determine whether the required expenditure is economically justifiable. Three types of highway improvement projects are analyzed and a typical example of each case is presented. These three types are:

1.—Improvement of a highway by the construction of a roadway surface of a higher type, including the necessary supplementary work.

2.—Improvements by means of relocations that shorten the distance between terminals.

3.—Improvements that reduce the rate of grade, with or without a change in distance.

While it is recognized that certain types of highway improvement present themselves, in which the dominating purpose is the removal of the accident hazard, the major portion of the construction work of State Highway Departments and of cities consists of projects that should go forward only when they are justifiable on economic grounds. Strictly speaking, the removal of the accident hazard is economically justifiable, but the analysis of such cases presents many difficulties, and will not be dealt with in this paper.

Basis of the Analysis

The criterion by which to determine whether a proposed highway improvement is economically justifiable is one of annual costs. In certain cases highway improvement may result in a lower annual cost of maintenance, using the term, "maintenance," in the broad sense of including all the cost incident to the perpetual maintenance of the highway in a standard serviceable condi-

Note.—Presented at the meeting of the Highway Division, Atlantic City, N. J., October 10, 1932. Discussion on this paper will be closed in December, 1933, *Proceedings*.

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tion. The greater part of major improvements, however, do not result in a reduction of the annual cost of highway maintenance and must be considered on broader grounds, in order to justify the necessary expenditures.

The principle that the ultimate purpose of highway improvement is to permit highway transportation at the minimum cost must be introduced into the analysis in most cases; that is, the improvement should result in a lower annual cost of transportation if it is to be justified on economic grounds. The cost of transportation consists of two elements—the road cost and the vehicle cost—and while certain types of improvements may actually increase the annual road cost, they decrease the annual vehicle operating costs sufficiently to more than offset the increased road costs. The net result is transportation for the traffic known to use the highway, at a cost less than that incurred before the expenditure for the highway improvement.

In a measure, the production idea may be introduced into road building. In manufacturing, the efforts of management are directed toward the production of a particular commodity at the minimum cost. Improved machinery or changed methods of production are introduced only after an exhaustive study to determine their effect on the cost of production, and if that cost is not decreased by the proposed changes sufficiently to justify the necessary expenditure, the old methods are continued in use. In highway building, a somewhat similar criterion may be set up. The actual cost of transportation on an existing highway may be ascertained with reasonable accuracy. The probable cost of transportation after a proposed improvement has been completed may be estimated within reasonable limits. If the correct method of analysis is followed and if values are obtainable for all the required factors, the results indicate quite conclusively whether or not the expenditure should be made.

The fundamental theory involved in the computations that will be used in any analysis of the character under discussion will be set forth briefly. Those competent to judge in matters of this nature are in substantial agreement with the material that is presented herein.

The starting point in the development of the necessary theoretical basis for economic comparisons in highway work is the simple statement that:

$$\text{Highway transportation costs} = [\text{Highway costs}] + [\text{Vehicle costs}]$$

The application of the principle thus set forth to a specific case requires that each of the groups of costs be ascertained and that they be reduced to some common unit so that they can be added to give a correct unit cost of transportation. Probably the most convenient unit would be the ton-mile, which is used in railway practice, but the nature of highway traffic does not readily lend itself to the use of ton-mile costs. The use of the vehicle-mile has developed gradually as a measure of highway traffic, and although such a unit is open to criticism from several standpoints, it will be used in this paper simply because it is already well established and generally understood. The general definition for the cost relationships needed for economic studies of highway transportation may be expanded somewhat, as follows:

$$\text{Comparative cost of highway transportation per vehicle-mile on a specified highway} = \left[\text{Highway costs per vehicle-mile of traffic}\right] + \left[\text{Cost of vehicle operation per mile of travel}\right] - \left[\text{Contribution by vehicle to road funds}\right] \quad . (1)$$

The highway costs that must be considered are all the costs of all kinds incident to the construction, maintenance, and administration of the highway. This may be expressed, as follows:

$$\text{Annual highway costs} = \left[\text{Interest on the investment}\right] + \left[\text{Annual cost of routine maintenance}\right] + \left[\text{Annual administration costs}\right] + \left[\text{Annuity for depreciation}\right] + \left[\text{Annuity for periodic repairs at intervals of } n' \text{ years}\right] + \left[\text{Annuity for periodic repairs at intervals of } n'' \text{ years}\right] + \ldots\ldots (2)$$

It should be obvious that Equation (2) may be converted to any unit desired; but for most purposes it will be found convenient to reduce these costs to annual highway cost per mile, per unit of traffic.

In any discussion of the annual cost of highways the question arises as to the application of interest to the computations. The position taken in this presentation is that proper account of interest must be introduced if strictly correct conclusions are to be drawn. The facts that no interest is actually paid out on funds used currently, and that interest is paid out when improvements are financed by bond issues, seem to be confusing to those who have studied this subject. As a matter of fact, it is of no significance in this connection whether the improvement is financed by current funds or by a bond issue. If the public finances a project from current funds, it has invested a certain sum in a utility from which it expects to obtain a service in lieu of a direct financial return. Had that sum remained in the hands of the individuals who comprise the public, each might have invested it in a dividend or interest-paying security. The highway, therefore, should be charged with interest on that part of the investment that represents deferred service and at the rate of interest the funds could undoubtedly earn if gainfully invested. That is exactly the rate the public must pay for money borrowed, since Government bonds constitute the safest investment available to the public.

When the financing is through a bond issue, the interest charged in computing the cost of highway service is actually paid out, but the public does not make its contribution to the cost of the road until a bond is paid and then interest ceases. Meanwhile, each individual may gainfully invest the money he will contribute later to the road-bond fund and if he is thrifty he can cause the money to earn as much interest as he will later pay; for example, he may purchase a road bond.

For these reasons the interest factor is introduced in the formulas set up herein, and in those applying to road costs the rate of interest is that borne

by road bonds, which is taken at 4%, but may be some other rate in specific cases.

The relationships set up in Equation (2) may be written in the form of an algebraic equation and, for purposes of illustration and to simplify the presentation, an equation applicable to the road surface only is, as follows:

$$C = Ir + \frac{(I - S)\,r}{(1+r)^n - 1} + M + A + \frac{E'r}{(1+r)^{n'} - 1} + \frac{E''r}{(1+r)^{n''} - 1} + \ldots (3)$$

in which,

C = the annual cost of any section of highway surfacing;
I = the initial cost of the surfacing, including engineering and administrative overhead;
r = the rate of interest adopted for the analysis (expressed as a decimal);
S = the residual or salvage value of the road surface when it is superseded by another type;
M = the annual maintenance cost for routine work that is paid for annually;
A = the annual administrative costs chargeable to the highway in question;
E' = the cost of periodical maintenance that is performed and paid for at intervals of n' years; and
E'' = the cost of periodical maintenance that is performed and paid for at intervals of n'' years. (This factor applies to improvements of a different character from those designated by E').

In all ordinary cases the factor, A, may be omitted from Equation (3) without significant error. It would be included in special cases, such as the Holland Tunnel and certain toll bridges.

A little consideration of Equation (3) will disclose that it provides a means of estimating the annual cost of the maintenance of a road surface in standard condition for a long period in the future as well as of determining the historical cost of a specific road surface.

An equation of like form can be used for estimating the annual cost of each of the additional elements of a highway, such as drainage structures, retaining walls, elevated structures, tunnels, earthwork, guard fences, and warning and direction signs. However, it will be found that some of these elements have such great life expectancies that it will introduce no appreciable error if they are considered to have unlimited life under appropriate maintenance. Other elements, such as guard fences, may be considered to have unlimited life under proper maintenance even if each unit may be completely replaced piece by piece over a comparatively few years.

Having determined the annual cost, C, of any section of highway, including all its elements, that cost, by obvious methods, can be reduced to C_a, the annual cost per mile per unit of traffic. This is comparable to the toll that would have to be exacted if the highways were private enterprises and were operated to give the investors a return at the rate of 4% per annum.

The cost of vehicle operation may be derived by a process analogous to that used in estimates of highway costs, and need not be discussed in detail herein.

The foregoing discussion has been limited, for simplicity, to a single element of the road surface and a single vehicle. For economic comparisons it is necessary to take into account the costs of all elements of the highway under consideration and of all those that accrue to the traffic, and to forecast the traffic and expenditures on the highway during a reasonable period in the future.

Future highway costs can be estimated with considerable accuracy, but the estimation of the various elements of vehicle-operating costs presents a most illusive and troublesome problem. Vehicle design, tire durability, fuel and oil costs, insurance rates, wages, mileage of operation, and speed of operation, are all factors in the problem. The cost of each of these elements is exceedingly variable in any community, and it might seem hopeless to make any analysis that would be of significance. However, by making careful comparative measurements with a few vehicles it has been possible to set up relative cost figures that certainly apply to test vehicles. It is probable that the same general relationships apply to all traffic although the unit costs vary widely. Studies of this subject have been under way at several places for a number of years and by piecing together the bits of information gleaned from these various researches certain relative figures on the cost of vehicle operation have been set up.

In order to discuss the relative costs of transportation on various kinds of road surfaces it seems expedient to use the grouping now so widely adopted by highway engineers, namely, high type, intermediate type, and low type. While this is convenient for a general discussion, it is important to recognize that road surfaces cannot be classified by name only, since there is great diversity in the physical characteristics of any one of the road surfaces as it is constructed in various parts of the United States. Therefore, the types must be grouped on the basis of their relative physical characteristics.

On the basis of the foregoing classification certain estimates of the relative cost of vehicle operation were developed and published in 1928, as an outgrowth of a series of investigations of the various factors involved. The significant results[2] are given in Table 1.

From a recent examination of data now being developed by the writer, he has reached the conclusion that, for economic studies, it is on the side of conservatism to assume that the change from one type to the next higher, as these are defined in this paper, saves to the automobile about one-eighth of the operating cost on the lower type. This amounts to 0.8 cent per mile of travel for the "average" automobile on the basis of 1931 operating costs.

For other classes of vehicles, particularly for trucks, it is necessary to deal with the actual composition of the traffic on a highway rather than with an average operating cost figure, since the character of the equipment in use in different parts of the United States is exceedingly diverse, and the composition of the commercial vehicle traffic reflects many local factors. The

[2] "Operating Cost Statistics of Automobiles and Trucks", by T. R. Agg and H. S. Carter, Members, Am. Soc. C. E. *Bulletin 91*, Iowa Eng. Experiment Station, Ames, Iowa, 1928.

data at hand are too fragmentary to permit drawing any very positive conclusions as to the saving to commercial vehicles through change in type of road surface.

It is believed that no serious error will result if economic comparisons are based on a value of 17 cents per vehicle-mile of travel as the average operation

TABLE 1.—RELATIVE COST OF OPERATING AN IMAGINARY "AVERAGE" AUTOMOBILE ON VARIOUS CLASSES OF ROADS

Item of cost	APPROXIMATE RELATIVE COST OF OPERATION, IN CENTS PER MILE		
	High-type roads	Intermediate-type roads	Low-type roads
Gasoline	1.09	1.31	1.61
Oil	0.22	0.22	0.22
Tires and tubes	0.29	0.64	0.84
Maintenance	1.43	1.72	2.11
Depreciation	1.26	1.39	1.57
License	0.14	0.14	0.14
Garage at $4 per month	0.44	0.44	0.44
Interest at 6%	0.36	0.36	0.36
Insurance	0.21	0.21	0.21
Total cost	5.44	6.43	7.50
Relative cost	1.00	1.18	1.38

cost of commercial vehicles on paved roads, and a saving in cost, by a change from an intermediate type to a high type, of 5 cents per vehicle-mile of travel. This saving results from lowered operating costs and an allowance for the value of the increased average speed that is possible. It is also believed that commercial vehicles with a capacity of less than 1 ton should be counted as automobiles, and that the commercial vehicle traffic should be considered as made up of commercial vehicles having capacities of 1 ton, or more.

In determining the allowance that should be made for decreased operation costs when a relocation shortens the distance between objectives, account must be taken of the fact that a shortening of distance does not affect the fixed charges incident to vehicle operation. A study of Table 1 would indicate that distance costs about 2½ cents per mile on high-type surfaces and 3.3 cents per mile on intermediate-type surfaces, if it is assumed that one-third of the maintenance and depreciation is due to the mileage of travel and the remainder is due to weather and obsolescense. With commercial vehicles the shortening of distance also accomplishes a decrease in the time of travel between objectives, and it is believed that a reasonable allowance for the shortening of a high-type surface would be 8 cents per mile and for intermediate types, 13 cents per mile.

ANALYSIS OF SAVINGS RESULTING FROM CHANGE OF ROAD SURFACE TYPES

There is next presented an analysis of the change in cost of transportation resulting from changing the road surface on a section of U. S. Highway 65 from an ordinary gravel surface to a high-type paved surface. The traffic

at the date of the change consisted of 700 000 automobiles per annum and a negligible amount of commercial vehicle traffic. The data used in this example are those on record for this particular highway, which is 26 miles in length. The investment data as of July, 1930, are first presented, and are followed by an analysis of the annual cost of transportation on this road while it was still a gravel surface.

As an intermediate type the investment in Highway 65 was, as follows:

Right of way	$38 000
Drainage structures	132 950
Earthwork	204 000
Road surfaces (light gravel)	84 700
Signs and other appurtenances	2 000
Engineering and administration, 4.5%	20 775
Total investment for 26 miles of road	$482 425

Annual Maintenance Cost.—The annual maintenance cost, including proper allowance for supervision, averaged $21 500 per year over a period of five years.

Periodic Maintenance.—The periodic maintenance consisted in re-shaping ditches and shoulders and re-graveling at intervals of three years at a cost of $34 300, which is E' of Equation (3). Since the time interval is so short it is really unnecessary to adopt the annuity method of determining the annual cost of periodical maintenance, but for illustrative purposes that method is followed herein. The annual cost of periodical maintenance is, therefore:

$$\frac{34\,300 \times 0.04}{(1.04)^3 - 1} = \frac{34\,300 \times 0.04}{1.125 - 1} = 34\,300 \times 0.32 = \$10\,980$$

Depreciation.—The total original investment in the gravel surface must be charged against the use of the gravel road since the salvage value of the gravel was zero when the high-type surface was laid. The depreciation charge is computed by the second term of Equation (4), which is, $\frac{(I - S)\,r}{(1 + r)^n - 1}$. In this case, $r = 0.04$ and $n = 8$ years. The annual cost of depreciation is, therefore,

$$\frac{84\,700 \times 0.04}{1.04^8 - 1} = \frac{84\,700 \times 0.04}{1.37 - 1} = 84\,700 \times 0.108 = \$9\,150$$

The value of A cannot be determined for the road in question. The annual road cost is determined by applying Equation (4), as follows:

$$C = 482\,425 \times 0.04 + \frac{84\,700 \times 0.04}{1.04^8 - 1} + 21\,500 + 10\,980 = \$60\,927$$

The length of the road is 26 miles and the annual cost per mile of road is, therefore, $2 345.

The annual road cost after conversion to a high-type surface will next be computed. This road was improved with a pavement, signs were installed, guard fences were constructed, and the highway was completely modernized. As a high type the investment in Highway 65 was, as follows:

Right of way	$38 000
Drainage structures	140 927
Earthwork	216 240
Prior surfaces (gravel having no salvage value)	0
Pavement	817 260
Signs and other appurtenances	4 450
Total	$1 216 877

Engineering and administration costs were included in the foregoing items, as they were incurred.

Annual Maintenance Cost.—The annual maintenance cost has been determined from official reports to be an average of $24 380 per year. This includes all the expenditures for maintenance of all elements of the highway.

Periodic Maintenance.—In this case, the need for any periodic maintenance has not occurred as yet. Based on the present forecast of probable life for this highway and the contemplated and feasible methods of prolonging the life of the investment, resurfacing of the concrete pavement may be effected at some future date, say, at the age of 20 years. At that time the existing road surface will serve as a base for a new surface and will have value as such. Assume that value to be $1.50 per sq yd, or $463 500, which is the factor, S, in Equation (3). The cost of resurfacing is forecast at $275 000, which is the factor, E', of Equation (2). The value of n in Equation (3) is 20 years. The data are now assembled for the application of Equation (3) to the calculation of the annual cost of this highway, as follows:

$$C = 1\,216\,877 \times 0.04 + \frac{[(1\,216\,877) - (463\,500)]\,(0.04)}{(1.04)^{20} - 1} + 24\,380 + \frac{(275\,000)\,(0.04)}{(1.04)^{20} - 1}$$

in which, $C = 48\,675 + 25\,298 + 24\,380 + 9\,235 = \$107\,588$. The annual road cost per mile (length = 26 miles) is $4 140.

The contribution to road funds, through the medium of the gas tax and license fee, by the automobiles using the gravel road, amounts to $1 660 per mile of road per year and by those using the concrete highway it amounts to $1 570 per mile of road per year. The cost of transportation may now be computed from Equation (1) as follows:

I.—As a Gravel Road:

Comparative cost of transportation per vehicle-mile:

$$\frac{2\,340 + 45\,010 - 1\,660}{700\,000} = 6.53 \text{ cents}$$

II.—As a High-Type Road:

Comparative cost of transportation per vehicle-mile:

$$\frac{4\,140 + 38\,080 - 1\,570}{700\,000} = 5.80 \text{ cents}$$

The investment, therefore, is justifiable on the basis of the saving in transportation costs to the traffic actually using the highway.

Estimating Value of Decreased Distance

To illustrate the method of estimating the value of decreased distance the following example is presented. Suppose that the average annual traffic on a highway consists of 1 500 000 automobiles and 100 000 commercial vehicles, that a relocation is arranged that saves a distance of 0.6 mile, and that the pavement costs $25 000 per mile. The annual savings due to decreased distance would be as follows:

Annual savings to automobiles	=	$1 500 000 × 0.025 × 0.6	=	$22 500
Savings to commercial vehicles	=	100 000 × 0.08 × 0.6	=	4 800
Total			=	$27 300

If it may be assumed that this relocation and consequent shortening of distance will be effective for the life assumed for the road surface, that is, 20 years, the present value of the annual saving of $27 300 is $371 000. To this would be added the $15 000 saving in cost of construction, making the present worth of the future service to be delivered by this relocation, $386 000.

Comparisons of this type should be used with great caution, however, because there is no assurance that the shortening of distance between two terminals, as was done in this case, will actually result in lowering the distance traveled by an automobile. To a considerable portion of the traffic there is this actual saving, but to another portion there is no such saving. It is a potential saving, however, and affords a means of forming a judgment as to whether a certain improvement is economically justifiable. In many cases the conditions surrounding a project are such that it is possible to determine with a considerable degree of accuracy the number of vehicles that will actually profit by the decreased distance.

Grade Reductions

It is doubtful whether there is any more elusive problem in the field of highway economics than that of estimating the potential saving that will result from grade reductions. Minor grade reductions may be accomplished by cutting and filling without relocation, but major grade reductions, especially on long hills, are usually accomplished by a combination of relocation and cutting and filling. The changes in grade that can be accomplished by cutting the summits and filling the sags without relocation usually have little economic value although exceptions may be encountered.

This problem is complicated still further by the fact that the operating characteristics of the vehicles that comprise road traffic are so variable that there is no such thing as a single economic grade. The engineer is forced to

compromise on grades that are feasible within the limits of the topographical conditions surrounding a project. A practicable and rational approach to this problem may be accomplished, as follows:

1.—Determine the total rise and the total station length of the grade which it is proposed to reduce.

2.—Determine the corresponding two factors for a proposed grade improvement which will permit ascent in high gear for most of the traffic.

3.—Assume 4% to be a safe coasting grade for the vehicles that use the road, and determine the excess rise on the proposed plan over that which would be provided by a 4% grade of the same length.

4.—Estimate the annual tonnage of traffic using this road. A conversion factor can be applied to the estimated volume of traffic to convert it to a tonnage basis. For most projects it is safe to assume that this traffic is divided equally between the two directions.

For purposes of these computations it may be assumed that a gallon of gasoline weighs 5.9 lb, that each pound of gasoline tests 19 000 Btu, that each British thermal unit is equivalent to 777 ft-lb, and that the average thermal efficiency of the engines in the vehicles that make up the traffic is 15 per cent. With this as a basis, the various relationships involved may be set up in the form of an equation for computation purposes, as follows:

Let H = height, in feet, of existing summit above the sag;
H_1 = height, in feet, of proposed summit above the sag;
H_2 = height, in feet, of summit of a 4% grade of stationing equal to proposed grade;
M = low-gear factor for the existing grade;
C = cost of gasoline, in decimal of a dollar per gallon;
S = annual savings by reducing grade; and
T = annual tonnage of traffic over highway.

Then:

$$S = \frac{2\,000\,[\frac{1}{2}T\,(M\,H - H_1) + \frac{1}{2}T\,(H_1 - H_2)]\,C}{19\,000 \times 777 \times 0.15 \times 5.9}$$

$$= 0.0015\,C\,T\,[(MH - H_1) + (H_1 - H_2)] \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The foregoing computations will show the annual saving by the grade reduction. If this grade reduction is accomplished by increasing the length of the road, then the saving computed by Equation (6) must be reduced by the extra cost of distance. This extra cost of distance can be computed as was outlined for computing decrease in distance.

A word with reference to the factor, M, which is designated as the low-gear factor for the existing grade, may not be out of place. If an existing grade were such that a vehicle could ascend in low gear, there would be no economic reason to change that grade unless the change resulted in a saving in transportation cost. As a matter of fact such a saving does result, due to the decreased time required for the ascent, and to the decreased cost of vehicle operation when the ascent can be made in some gear faster than "low."

Therefore, the factor, M, represents the quotient obtained by dividing the cost of ascending an existing grade with the gear that would have to be used, by the cost of making the ascent if it could be made in high gear, at high-gear speed and high-gear operating efficiency. It can only be determined experimentally.

Example of Computation of Value of Grade Reduction

Let it be assumed that on a surfaced road (Fig. 1) an existing grade 2 000 ft long has an average 12% grade, and, consequently, a rise (H) of 240 ft, and that it is to be changed to a 7% grade which will require a relocation that will give 3 000 ft of grade with a rise (H_1) of 210 ft. The equivalent of 3 000 ft of 4% grade would have a rise (H_2) of 120 ft. Suppose that

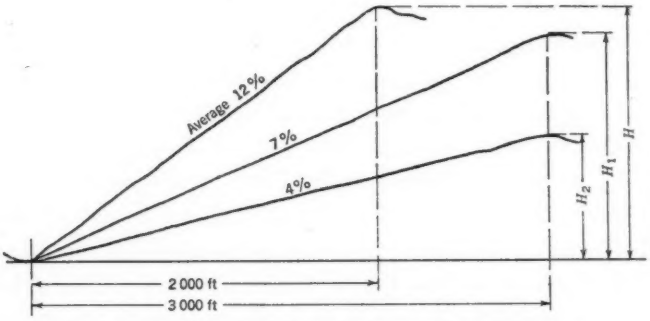

Fig. 1.—Illustrating Economics of Grade Reduction.

the annual traffic over this grade is estimated at 4 000 000 tons and that the average price of gasoline is 19 cents per gal. The factor, M, in this case may be taken at 1.4. The annual saving by this grade reduction, therefore, would be, as follows:

$$S = (0.0015)(2\,000\,000)(0.19)\ \{[(240\times 1.4)-210]+[210-120]\} = \$123\ 120$$

Let it be assumed that the 4 000 000 tons of traffic using this highway consist of 3 000 000 automobiles at an average weight of 1 ton, and 250 000 trucks at an average weight of 4 tons. The cost of the extra travel to the 3 000 000 automobiles, at $2\frac{1}{2}$ cents per mile, would be \$75 000 per year. The cost of the extra travel to the 250 000 trucks, at 5 cents per mile, would be \$12 500 per year, or the total cost to the traffic of the extra travel made necessary by lengthening this grade would be \$87 500. The saving to traffic on account of the grade reduction was \$123 120. Deducting from this the extra cost of the extra distance, or \$87 500, leaves the net value of the saving to traffic by this grade reduction as \$35 620 per annum. This may be capitalized on whatever basis seems appropriate, but in view of the conditions that surround the highway transportation field, it is well to be conservative. Suppose it were to be capitalized on a 4% basis, and amortized within a period of 20 years. The expenditure that could be justified on economic grounds for this work would then be the present worth of an annuity of \$35 620 continuing for 20 years at 4% rate of interest, or about \$485 000.

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PAPERS

THE SURVEYOR AND HIS LEGAL EQUIPMENT

BY A. H. HOLT,[1] M. AM. SOC. C. E.

SYNOPSIS

There are many places at which the work of the Engineer and that of the Lawyer overlap, but possibly there is no department of his professional activities in which the Civil Engineer has more specific need to use either his own or another's knowledge of the law than in land surveying. Especially is this true in the re-establishment of lost or disputed boundaries.

To some persons a complete answer to the problem presented by a lost or disputed boundary seems to be to call in a surveyor to "run the line." If a "starting point" is available, even if it is remote from the boundary to be located, a common impression seems to be that a competent surveyor should have no uncertainty in how to proceed, nor experience any real difficulty in arriving at a definite solution. In fact, the result is scarcely to be thought of as a "solution." The surveyor should be able, it seems, by the use of his instruments alone, to establish the boundary with certainty, and with assurance that, thus established, it will stand. A larger group would probably be willing to assert that when the dimensions—courses and distances—of a boundary are clearly evident, perhaps on a plat, any consideration of the location of the boundary by other means than the careful running out of the description on the ground would be idle.

If through earlier mistake or carelessness that which may long have been accepted as the boundary does not agree with that which is now precisely located, the sooner the change is made the better. This attitude toward their abilities and powers may possibly be complimentary to surveyors; but to attempt to measure up to its demands a surveyor must either be unencumbered by the possession of a conscience, or unhampered by experience.

It is desired to speak of two aspects of boundary location—very briefly of its importance, and somewhat more at length of some of the factors which

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control or influence it. Those who are interested in studying this general subject further are referred to books written by Ray H. Skelton,[2] M. Am. Soc. C. E., and Mr. Frank E. Clark.[3]

A Fundamental Proposition

A fundamental proposition should be stated at the outset: That the proper location of a boundary is that which a competent and well-advised Court would decree. The only exception to this rule is a location on which the owners concerned may agree for themselves. This they may always do validly, as long as no other interests are thereby jeopardized.

That the proper location of a boundary is a matter of some consequence, that failure in this respect can be as costly as shortcomings in engineering design, the decisions of the Courts of last resort of the United States are ample testimony. In the system of law that governs this country, drawn from its heritage of feudal customs and traditions, fostered by its development through a period in which, it has been said, the law of England was framed to help the King of England run his farm, no property rights are given a higher grade of protection than is given to rights in land. The name, "real property", is not without meaning.

One of the most fundamental of these rights is that of exclusive possession. If one of two co-terminous owners, because of an erroneous conclusion as to the location of the common boundary, extends his building on to his neighbor's land, there is nothing left for him to do, if the neighbor cares to insist, but to remove it and, in addition, to pay damages for the intrusion. He cannot, through his own wrong (the trespass) force his neighbor to sell the strip of land to him. True it is, that in a very few cases an Equity Court has restrained the execution of a judgment of ejectment, or has refused an injunction; but only when the action of the invader has appeared in a favorable light, and it has been shown that there is no substantial damage to the use of the land invaded. One trial Court, for example, finding that the trespass was to the extent of $\frac{1}{2}$ in. beyond the boundary, refused to require the removal to that extent of an underground concrete foundation. A higher Court, however, reviewing the evidence on appeal, concluded that the intrusion was over a strip about 3 in. wide, and decreed its removal. The cost of complying would have paid, many times over, for an accurate survey.

A prominent building in New York City was found to have been built beyond the line. The owners, in this case, found it possible to persuade the adjacent owner to sell to them the 6-in. strip involved. The cost of the persuasion is stated to have been $50 000. This was less than the cost of removing the building, but an accurate survey would have saved it.[4]

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[2] The Legal Elements of Boundaries and Adjacent Properties" by Ray H. Skelton, The Bobbs-Merrill Co., 1930.

[3] A Treatise on the Law of Surveying and Boundaries" by F. E. Clark, The Bobbs-Merrill Co., 1922.

[4] Mulrein *v.* Weisbecker, 37 App. Div. 545, 56 N. Y. S., 240; Machstein *v.* Christopher, 128 Ga. 229, 57 S. E. 511, 11 L. R. A. (N. S.), 917; Murphy *v.* Bolger Bros., 60 Vt. 723, 1 L. R. A. 309, 15 Atl., 365. However, see Harrington *v.* McCarthy, 169 Mass. 492, 48 N. E., 278; Methodist Episcopal Soc. *v.* Akers, 46 N. E. 381, 167 Mass., 560; Rahn *v.* Milwaukee Elec. Ry. & Lt. Co., 103 Wis. 467, 79 N. W., 747.

The difficulty in each of these cases arose from lack of appropriate precision in the execution of a survey. It is unnecessary to discuss the impossibility of an exact survey; and a Court cannot do otherwise than accept a reasonably precise location in lieu of one as exact as the language of lawyers and of Courts might seem to require. Is it sufficient, then, for the safeguarding of the interests of clients, that a surveyor be capable of making surveys accurately and with all the precision which is reasonable and appropriate for the property concerned? The answer of an experienced and honest surveyor to this question would probably be that he can do the surveying well enough if he can only make certain what should be done. It is this determination of what factors should influence or control the location of the boundary, which requires a surveyor, in order to render to his client service of a maximum value, first to function as a counsellor-at-law—perhaps it is not too much to say, in some respects, as a Court.

Without in any way minimizing the importance of precise surveying, it may be said that in the re-establishment of uncertain or disputed boundaries the use of surveying instruments or calculations is but one means—although an extremely valuable means—of assembling evidence, or of giving effect to a decision, as to where the line should be.

The comment may perhaps be made that a surveyor should be concerned only with surveying—not with interpretation of evidence or application of law—that his activities should be restricted to a survey and a report to his client, leaving the client to do with them as he thinks best. A surveyor who adhered to this narrow path would be a tradesman indeed. As a practical matter, the client usually expects the surveyor to determine where the line should go and depends upon him to do so. The client does not desire a report which will require him to seek the counsel of a lawyer as to how to proceed. Thus, it is that in order to render to his client the service that the occasion frequently demands, a surveyor, called in to "run a line," may need to possess and use a sound knowledge of the law of boundaries. Lacking this, or lacking experience and judgment in its application, he may easily become a cause of trouble between adjoining owners, instead of fulfilling his proper function of justly and judicially disposing of the difficulty.

By a misguided survey, even though it may be an accurate one, he may aggravate to the point of Court action a situation which with proper handling would have been settled amicably and economically; or, if the case be one that must be settled in the Courts, he may bring his client into Court on an improper, and therefore a losing, ground. The writer is acquainted with more than one case in which a mutual and friendly desire on the part of adjoining land-owners to have their common boundary properly located was turned into a dispute by a needless or an inaccurate survey of the line by the surveyor. In one such instance the proper location was finally reached only after four surveys, much hard feeling, an erroneous decision in the District Court, and a reversal by the Supreme Court of the State. Two days' work by a competent surveyor at the beginning should, and probably would, have ended the matter.[5]

[5] Meyers *v.* Wonick, 180 Ia., 286.

A Cardinal Principle

There is one cardinal principle of the law of boundaries, applicable to this situation, which should never be disregarded, namely:

> A boundary line having once been authoritatively—even though inaccurately, or even erroneously—fixed in position on the ground, the true purpose of a re-survey is to reproduce that location; not to "correct," or otherwise change, it.[6]

No statute, no incumbency of office, can give to any person the right to "establish" a line, by precise survey or otherwise, in contravention of this principle, and to the disturbance of existing rights. If a boundary has once been authoritatively established, property rights have been vested in some one with respect thereto; and the mere obliteration or loss of the physical evidences or even of the record of the boundary, cannot divest those rights. Furthermore, a more precise re-running of the survey by which a boundary was once located cannot detract from, or add to, the property rights that have already accrued, and thus shift an established boundary.[7] From the more precise survey a more accurate description may perhaps be written, but the original boundary must stand.

Order of Calls

A surveyor's first duty, then, when called in to re-establish a boundary of which the location has become uncertain, or is in dispute, is the judicial task of assembling and evaluating the various available items of evidence of its original position. Unfortunately, there is likely to be some conflict or disagreement among these items, and a proper relative weight or importance must be assigned to them. This precedence of certain types of evidence of original boundary location is known as the "order of calls."[8] It must be continually kept in mind that the purpose to be accomplished is to recover the boundary as it was located originally; or, if it was not actually marked on the ground, then to carry out the intention, as effectively expressed, of those by whom or for whom the boundary was originally established or described. For this reason, those evidences of boundary which are most material, most certain, and least liable to error should prevail.[9] Any order of calls or other rules must be used and interpreted so as to further this purpose in the case in hand. They should never be applied arbitrarily when such application would obviously defeat it.[10]

Experience has shown that marks on the land itself, being capable of being seen by owners, and not being subject to errors of survey, computation, or description, are most likely to express the intent of parties. The lines and corners actually marked upon the ground by the parties concerned—by

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[6] Nesselrode *v.* Parish, 59 Ia. 570, 13 N. W., 746.

[7] Diehl *v.* Zanger, 39 Mich., 601.

[8] Note, 129 Am. St. Rep., 990.

[9] Newsom *v.* Pryor, 7 Wheat (20 U. S.), 7, 5 L. Ed., 382.

[10] Moran *v.* Lezotte, 54 Mich. 83, 19 N. W., 757, 759; Connor *v.* Johnson, 59 S. Car. 115, 37 S. E., 240; Pritchard *v.* Reborl, 135 Tenn. 328, 186 S. W., 121

the owners of adjoining tracts in establishing a boundary, or by a grantor in connection with a conveyance—will prevail, if they can be recovered, over all other calls, even over conflicting calls for more substantial monuments, to which the description may be referenced.[11] It is essential, however, that these marks of line or corner, in order to have this effect, be so named in the description of the deed or other document as to make it apparent that they actually expressed the intended location at the time it became effective.[12] This rule will prevail, also, when the discrepancy is with reference to a call for an adjoining boundary to which the survey is said in the description to run, if the survey was actually run and marked on the ground without reaching the named adjoining boundary.[13]

In case of conflict between marks placed upon the ground to indicate boundaries, and maps to which deed descriptions may refer for location of the same boundaries, the rule seems to be that "it is by the work as executed on the ground, not as projected before execution, or represented on the plan afterwards, that the actual boundaries are determined."[14] Although a deed may call for a plat only, not mentioning any survey—for instance, in transferring a numbered lot as shown on a certain plat—it has been held that a call for a lot according to a plat is a call for that which is only recorded on or represented by the plat, namely, the boundaries as marked in the field.[15] Marks of boundary will control over conflicting descriptions in terms of course and distance, and also over any call in a deed for a specified quantity of land conveyed.[16]

To be thus preferred these marks of boundary must have been made prior to or at the time of the grant, and it must be apparent that they were adopted as such (by reference in the deed, or otherwise) by the parties concerned; and the marks now found must be identified as those to which the reference is made, or else they cannot be regarded as expressing the intention of the parties.[17]

Recognizing that the boundaries actually marked by or for the parties concerned are to control if they exist and can be found and identified, and remembering that this and all other rules concerning evidence of the location of boundaries exist only for the purpose of discovering the intent of parties as effectively expressed, there remains the necessity of considering the relative value or effect of the various classes of evidence of boundary. The marks of boundary just mentioned will naturally fall into one or another of these classes. In this order of preference, natural monuments are given

[11] Jackson *v.* Miller, 195 S. W. (Mo.), 703; Thatcher *v.* Matthews, 101 Tex., 122, 105 S. W., 317; Messer *v.* Reginnitter, 32 Iowa, 312.

[12] Ritter Lbr. Co. *v.* Montvale Lbr. Co., 169 N. Car., 80, 85 S. E., 438, 443; Elliott *v.* Jefferson, 133 N. Car., 207, 45 S. E. 558, 64 L. R. A., 135.

[13] Matheny *v.* Allen, 63 W. Va., 443, 60 S. E. 407, 129 Am. St., 984.

[14] Ovens *v.* Davidson, 10 U. C. C. P., 302, 310; Rowell *v.* Clark, 119 Iowa, 299, 93 N. W., 280.

[15] Jackson *v.* Freer, 17 Johns. (N. Y.), 29, 31.

[16] Millerman *v.* Megaritty, 242 S. W. (Tex.), 757, 760; Arms *v.* Owatonna, 117 Minn., 20, 134 N. W., 298.

[17] Woodbury *v.* Venia, 114 Mich. 251, 72 N. W., 189; M. K. & T. R. R. Co. *v.* Anderson, 36 Tex. Civ. App. 121, 81 S. W., 781, 784; Currier *v.* Jones, 121 Iowa 160, 96 N. W., 766.

first place; artificial monuments are second; calls for adjoiners rank third; courses and distances, fourth; and quantity or area inclosed, last.[18]

Any physical landmarks adopted or established to indicate a boundary are regarded as "monuments."[19] While the classification of monuments as "natural" or "artificial" distinguishes, generally, works of Nature from those of Man, the purpose of the classification apparently causes some variation from this line of distinction. Obviously, streams, shore lines, trees, etc., are natural monuments;[20] and such structures as walls, well-built fences, and highways, although they are works of Man, have been so classed. The following definition is on record:[21]

"Natural monuments are objects permanent in character, which are found upon the land as they were placed by Nature, such as streams, lakes, and ponds, shores and beaches, also highways and streets, walls, fences, trees, hedges, springs, and rocks, and the like. * * * Wherever natural monuments, referred to in the description of a deed as marking the boundary, can be found, they fix the limits of the premises granted, though they correspond neither with the courses and distances nor with the quantity of the land given in the same descriptions."

Artificial monuments, for the purpose of assigning relative weight as evidence of boundary, are objects of lesser stability and permanence, or objects set specifically to mark points of boundary, such as stones, concrete markers, iron pipes or pins, etc.[22] Wooden stakes have been rejected by some Courts as monuments,[23] but if substantial they are usually acceptable.[24]

Calls for adjoiners are in a sense calls for monuments in that they require the land to extend to a line which exists, or may be brought into existence, on the ground, independently of the survey or description under consideration.[25] As such they are generally to be preferred to course and distance.[26] However, a large discrepancy might indicate a mistake in naming the adjoiner, or a misunderstanding as to where the line of the adjoining tract was, so that the preference would be reversed, especially if the called boundary is an unmarked line.[27]

Monuments and lines of adjoiners are actually boundaries, while courses and distances are at best only descriptions of boundaries. This being the case, these descriptions, or courses and distances, can control only (1) when there are no calls for actual boundaries; (2) when some mistake precludes the use of the actual boundaries which are specified; or (3) when these actual

[18] Meyer *v.* Comegys, 147 La. 851, 86 So., 307; Yanish *v.* Tarbox, 49 Minn. 268, 51 N. W., 1051.

[19] Koch *v.* Gordon, 231 Mo. 645, 133 S. W., 609, 610.

[20] Miles Land Co. *v.* Hudson Coal Co., 246 Pa. 11, 91 Atl., 1061.

[21] An American and English Encyclopedia of Law, Second Edition, p. 764.

[22] Sayers *v.* Lyons, 10 Iowa, 249.

[23] Bast *v.* Mason, 165 Mo. App. 718, 726, 148 S. W., 398; Cox *v.* Freedley, 33 Pa. 124, 130, 75 Am. Dec., 584.

[24] Tomlinson *v.* Golden, 138 N. W. (Ia.), 448; Thatcher *v.* Matthews, 101 Tex. 122, 105 S. W., 317.

[25] Bryant *v.* Maine Cent. Ry. Co., 79 Me. 312, 9 Atl., 736; Fidelity Realty Co. *v.* Flahaven Land Co., 193 Ky. 355, 236 S. W., 260.

[26] Land Co. *v.* Saunders, 103 U. S., 316; Graybeal *v.* Powers, 76 No. Car., 66, 69.

[27] Miller *v.* Grunsky, 141 Cal. 441, 75 Pac., 48; Jones *v.* Hamilton, 137 Ky. 253, 125 S. W., 695.

boundaries have become lost, not merely obliterated. Courses and distances are valuable to assist in finding or in identifying marks of boundary;[28] and they themselves may determine the boundary when no superior call appears, or when the superior call is found to be erroneous.[29] In the case of an inconsistency between course and distance some Courts appear to favor allowing the course to control. The following opinion[30] by the United States Supreme Court seems the better solution:

"It may be laid down as a universal rule that course and distance yield to natural and ascertained objects. But where these are wanting, and the course and distance cannot be reconciled, there is no universal rule that obliges us to prefer the one to the other. Cases may exist in which the one or the other may be preferred upon a minute examination of all the circumstances."

The distinction between a "lost corner" and an "obliterated corner," as the terms are used herein, is a fairly clear one. The reference is really to the physical object which once marked the point in question. If that object has disappeared, but if the position which it once occupied can be determined by any reliable extrinsic evidence (such as bearing trees, fence lines, or even the testimony of persons who have seen it), the corner is only obliterated, not lost; and it then has all the controlling effect that it would have had if the physical marker had remained. If not only the physical marker which once stood at the corner is gone, but there is insufficient reliable evidence to point out with reasonable precision the place it occupied, the corner is lost; and its control is gone. The arbitrary measures of surveying sometimes prescribed for the relocation of corners can apply only to situations in which the corners have become lost—never if they are merely obliterated.

Any corner which is "re-established" by any arbitrary method of surveying—such as, for example, a quarter-section corner re-established by marking the mid-point of a straight line joining the two adjacent section corners—is quite likely to occupy a position materially different from that originally marked. This new location, although it may conform to the requirements of statute, can have no effect in divesting any private rights of land ownership which can be shown to exist, even if the boundaries of such ownership were originally dependent upon the corner thus "re-established." Only in cases where the sole available control for such boundaries consists of descriptions dependent upon the surveyed line can this newly established corner determine old private boundaries.

Unless the quantity of land conveyed by a deed is made the essence of the contract, or unless there is a covenant to convey a specified amount of land,[31] the statement in the deed of the area conveyed has little effect in controlling boundaries:

"A definite description of lands in a deed designating the initial point and the courses and distances, followed by a statement of the number of acres

[28] Palmer *v.* Magers, 85 W. Va. 415, 102 S. E., 100, 103.
[29] Andrews *v.* Pitts, 126 Md. 328, 95 Atl., 203.
[30] Preston's Heirs *v.* Bowmar, 6 Wheat, (19 U. S.) 580, 582, 5 L. Ed., 336.
[31] Sanders *v.* Godding, 45 Iowa, 463, 465; Doyle *v.* Mellen, 15 R. I. 523, 8 Atl., 709.

conveyed, passes the quantity of land embraced in the specified boundaries, though greater or less than the number of acres stated."[32]

The words, "more or less," frequently follow the statement of quantity. Their purpose should be to show that all the land embraced within the specified boundaries is intended to pass by the conveyance, even though the actual area be somewhat at variance with that stated. However, these words should not so qualify the statement of the deed as to throw all risk of deficiency,[33] upon the grantee. In one case the grantee was granted relief where the deficiency was approximately 8 per cent.[34]

The foregoing suggests some of the principles of law that bear upon the recovery of an uncertain boundary. Their effect is first to require a careful assembling of all available items of evidence of the original location. Of prime importance in this process is an exhaustive search on the ground for the "footsteps of the surveyor," and for any other physical evidences of boundary. This should be no haphazard, superficial inspection; but a careful, methodical search. Consideration of this matter is quite outside the province of this paper; but any surveyor of experience is acquainted with instances of the futility of the former, and of the occasional success of the latter under seemingly hopeless conditions.

All evidence disclosed must then be weighed, in the light of the law as developed by the decisions, to ascertain the most probable location of the original lines. This law is simply a judicial expression of the results of long experience in working toward this purpose. The "running of the line"—that means by which the surveyor is frequently expected at once to produce a definite conclusion—will be done only in furtherance of and in conformity with this preliminary judicial consideration. To save time by a more hasty process is sometimes both disappointing and expensive.

Doctrine of Adverse Possession

The necessity may appear, however, of first determining whether the original boundary should or need be recovered at all; whether, on the other hand, it may be that by some development over a period of years another line may, through action of law, have been substituted for that which was once the boundary. One of two adjoining owners may occupy and use land beyond the line which surveys and deed descriptions would indicate as his boundary. If this is done in a way and for a length of time to satisfy the conditions of the law, the public interest in permitting him to remain peaceably in the situation in which he has established himself will induce the Courts to decide that he has acquired a prescriptive title to the land. The title of the old owner is then gone, because a new title has come into existence which is incompatible with the old. Had the old owner acted in time he could have prevented the acquisition of this new title, and thus have retained

[32] Syllabus by the Court, Pohlman *v.* Evangelical Lutheran Trinity Church, 60 Neb. 364, 83 N. W., 201; Dashiel *v.* Harshman, 113 Iowa 283, 85 N. W., 85, 87.

[33] Darling *v.* Osborne, 51 Vt., 148.

[34] Paine *v.* Upton, 87 N. Y. 327, 41 Am. Rep., 371.

his own; but it is now too late. A common boundary has thus been fixed by occupation, replacing the original one; and to re-run surveys or to attempt to execute the deed descriptions on the ground would be idle and of no avail. This is the American doctrine of "adverse possession"—a doctrine of affirmative prescription. In Iowa, this doctrine is almost inseparably connected with that of "acquiescence."

The decision as to whether prescriptive title has been acquired by adverse possession, or whether the conduct of the adjoining owners has by acquiescence established a boundary, is, in most cases, for the Courts; but it is essential that a surveyor be cognizant of these possibilities, and that he be acquainted with, and able to recognize, the items of evidence which tend to establish the existence of either of these situations. Thus, he may avoid a useless survey, and will be able to assist in assembling the evidence which will enable his client's attorney, or if necessary, a Court, to reach a proper decision.

The attitudes of the Courts in these matters have developed quite differently in the different States, and it will be best to limit this paper to a brief and general consideration of the law of one jurisdiction. Hence that which follows is based particularly on the law of the State of Iowa. A study of the cases is essential, of course, to any accurate or detailed understanding of the law of any jurisdiction.

To culminate in a prescriptive acquisition of title, it is necessary that the possession of the land by the invader be (1) actual, not merely constructive; (2) open and notorious; (3) under color of title or claim of right; (4) exclusive; (5) hostile to the interests to which the possession is alleged to be adverse; and (6) continuous for the period of the statute of limitations. In Iowa, this period is ten years.[35] Many pages of the reports of the decisions of American Courts have been occupied with defining these requirements, and determining whether the facts of specific cases measured up to them. Two or three of the cases in Iowa are outstanding in their development of the Iowa law concerning the situation with which surveyors are most concerned; that is, where the occupation has been by some one who has acted in good faith, with no purpose of acquiring land which did not belong to him, but who, because of mistake or misunderstanding, has for more than ten years occupied land beyond what now appears to be the surveyed or described boundary.

Doctrine of Acquiescence

An early case[36] established the position that the Iowa Court has since maintained, namely, that the belief of the possessor that he owns the land possessed does not amount to or take the place of the claim of right required by law. The possession under mistake is held not hostile to the interests of the true owner, because the possessor intended to occupy only that which belonged to him; and, consequently, it is not "adverse." That would appear to dispose of the matter in many cases in which a surveyor might be con-

[35] Code of Iowa, 1931, Section 11007.
[36] Grube *v.* Wells, 34 Iowa, 148.

cerned, leaving the ownership in the holder of the paper title; but a decision by Mr. Justice Ladd in 1900,[37] while it cited and re-affirmed the decision in the case of Grube *v.* Wells, laid down another closely allied doctrine which, in Iowa, supplements, almost to the point of superseding, the doctrine of adverse possession. That is the doctrine of acquiescence. In that case two adjoining owners had for many years occupied and cultivated up to a certain fence, treating it as their common boundary. A survey developed the fact that the fence was several feet from the surveyed boundary, which was a line of the Public Land Surveys. The Court stated:

"But, if it be conceded that this survey tended to fix the location of the government line as originally established, it does not follow that it should be regarded as the boundary between the coterminous owners. * * * But if the coterminous owners have adopted another line as their division line, and have occupied up to it and recognized it as such for a period of ten years, there apears to be no reason for not regarding it as the true boundary line, notwithstanding it is not that fixed by the government survey. * * * In the absence of other controlling circumstances, the inference is conclusive that the division line between adjoining tracts, definitely marked by the erection and maintenance of a fence or other monuments, recognized by the owners as such, and up to which they have occupied and cultivated the land on either side for more than ten years—the statutory period of limitations—is the true boundary between them."[38]

This decision is in accordance with a statute which was then Section 4236 of the Code of 1897, and is now Section 12306 of the Code of Iowa, 1931. Thus, it is seen that on the ground of adverse possession, or on that of acquiescence, boundaries between private owners may, by usage alone, be established in a fence or in a line otherwise marked. It is immaterial whether this line is that for which deed descriptions might call, or that which a retracing of Government or private surveys might locate; hence, the matter is one for consideration before time is spent in search or surveying. If a line has thus been fixed, a new survey may sometimes be profitably made to perpetuate it, but a re-survey cannot change it.

Unfortunately, the cases are not altogether clear in the matter of distinguishing adverse possession from acquiesence. It has been held that boundary lines may be established by recognition and acquiesence, independently of adverse possession, except for the element of ten years of time;[39] and that although the doctrine of adverse possession does not apply, in Iowa, to a case in which a party has had no intent to claim more than his deed calls for, a line may be established by recognition and acquiesence, although neither of the parties has intended to claim more than is described in his deed.[40]

It has been held that one who pleads that a certain line is the true boundary because it has been recognized and acquiesced in for the statutory period, must stand or fall on that theory; that he will not, after failing to

[37] Miller *v.* Mills County, 111 Iowa 654, 82 N. W., 1038.

[38] Miller *v.* Mills County, 111 Iowa, 654, 657, 659, 82 N. W., 1038, and cases there cited.

[39] Helmick *v.* D., R. I. & N. W. R. Co., 174 Iowa 558, 156 N. W., 736.

[40] Bradley *v.* Burkhart, 139 Iowa 323, 115 N. W., 597.

establish acquiescence, be permitted to prevail on the claim of adverse possession only.[41] Other cases, however, hold that one who for ten years has claimed title to and occupied premises up to a definite boundary line with the apparent acquiescence of the adjoining land-owner has acquired title either by acquiescence or else by adverse possession.[42] With this matter of distinction, however—important though it is to the lawyer and to the Court—the surveyor need not ordinarily be concerned.

From the standpoint of identifying the situation in which establishment of boundary by acquiesence may be the result, it is important to note the requisite acts, attitude, or knowledge on the part of the claimant (of land beyond the surveyed boundary) and on the part of the land-owner against whom the claim is made. Although the doctrine is often referred to as one of mutual acquiesence,[43] the acquiesence is, in reality, all on the part of the owner adjoining the claimant. The claimant must show, by clear and definite acts,[44] an intention to claim to a definite line [45] as the true one, which he must honestly believe it to be.[46] The mere establishing of a line,[47] or mere occupancy, is not sufficient. If this be acquiescence it is of a very positive character, and is very similar to that open and notorious possession required for adverse possession.

The acquiesence of the adjoining land-owner, on the other hand, may be of a passive, negative character.[48] It may be shown by his conduct, as by occupying up to the line[49]—although that is not required[50]—by words or by remaining silent when he had notice or knowledge of existing conditions.[51] It has been held, that where the land encroached upon is vacant and the owner absent, the doctrine is inoperative,[52] on the ground that actual knowledge on the part of the adjoining land-owner is essential to his acquiesence.[53] On the other hand, it has also been held[54] that the erection by the claimant of a house on a city lot, with reference to the survey stakes there existing, was notice to the world of her claim as to the location of the boundary, so that the adjoining owner and his non-resident grantors were held to have acquiesced in the boundary as such.

It is generally held that the doctrines of adverse possession and acquiescence do not apply to land held for public purposes by the Federal[55] or State

[41] Morrow *v.* Hall, 169 Iowa 534, 151 N. W., 482.
[42] Dake *v.* Ward, 168 Iowa 118, 150 N. W., 50.
[43] Taylor *v.* Olmstead, 201 Iowa 760, 206 N. W., 88.
[44] Griffin *v.* Werdell, 182 Iowa 969, 164 N. W., 760; Lynch *v.* Northwestern Laundry, 194 Iowa 317, 189 N. W., 748.
[45] Quinn *v.* Baage, 138 Iowa 426, 114 N. W., 205.
[46] Keller *v.* Harrison, 139 Iowa 383, 116 N. W., 327.
[47] Webster *v.* Shrine Temple Co., 141 Iowa 325, 117 N. W., 665.
[48] Dake *v.* Ward, *supra.*
[49] Morley *v.* Murphy, 179 Iowa 853, 162 N. W., 63.
[50] Kotze *v.* Sullivan, 231 N. W. (Ia.), 339.
[51] C. M. & St. P. Ry. Co. *v.* Hanken, 140 Iowa 372, 118 N. W., 527; Dwight *v.* City of Des Moines, *infra.*
[52] Palmer *v.* Osborne, 115 Iowa 714, 87 N. W., 712.
[53] Dwight *v.* City of Des Moines, 174 Iowa 178, 156 N. W., 336, 339; Morley *v.* Murphy, *supra.*
[54] Helberg *v.* Kepler, 178 Iowa 354, 159 N. W., 972.
[55] Stanley *v.* Schwalby, 147 U. S. 508, 514, 37 L. Ed. 259, 13 Ct., 418.

Governments, or by municipalities.[56] A private owner who fails for a long period of time to exercise his right to eject an invader, finds that that right has been cut off. The public can act only through its agents, the elected or appointed officers; and public interests should not be permitted to suffer through their unauthorized neglect. It may be noted in passing that if, through positive acts of a qualified representative of a municipality, a private owner is led to occupy land beyond his proper line, and into that which should rightfully be devoted to public use, the doctrine of equitable estoppel may come into play to prevent the municipality from so disavowing the acts of its representatives as to require the private owner to withdraw.[57]

Establishment of Boundaries by Agreement

Acquiesence does not presuppose an agreement to a line; but, on the other hand, agreement to the boundary is to be inferred from long acquiesence therein.[58] However, the establishment of a boundary by agreement may often be the most economical, most sensible, and most satisfactory means of settling the location of an uncertain or a disputed boundary. Authority that such an agreement, properly consummated, will be effective and binding may be found both in the decisions and in the statutes.[59] It is not intended that this process of agreement on a boundary should be used to effect a change in the position of a known boundary, thus resulting in the transfer of land without the formality of a deed;[60] although it has been held that an oral agreement to change a long established boundary is enforceable when taken out of the statute of frauds by (1) the mutual taking of a new survey; (2) the building of a new fence in accordance with the said survey; and (3) the taking possession of the lands inclosed by such fence.[61]

It frequently happens, however, that a surveyor is called upon to relocate a line for which the control is difficult to find, expensive to reach, and of very doubtful certainty or precision. The general location of the line is usually known to the adjoining owners. If they would forego the survey, and establish the line by compromise and agreement, or permit the surveyor as an arbitrator to make such a compromise location, the loss to either owner in value of land involved between a location thus made and the extreme position for which he might contend, would frequently be far less than the cost of the search and survey. The compromise location having been agreed upon, the line may be monumented, surveyed, tied in to enduring control, and platted. This procedure, followed by a formal and recorded acceptance of the line by adjoining owners as their boundary, will fix that line as such for all time. The saving in dollars is often considerable, but the saving in

[56] Schultz *v.* Stringer, 168 Iowa, 675; Quinn *v.* Baage, 138 Iowa, 426, 114 N. W., 205; Johnson *v.* City of Shenandoah, 153 Iowa 453, 133 N. W., 761; Ralston *v.* Town of Weston, 46 W. Va. 544, 33 S. E., 326.

[57] Krause *v.* El Paso, 101 Tex. 211, 106 S. W. 121, 14 L. R. A. n. s., 582.

[58] Keller *v.* Harrison, *supra.*

[59] Koppes *v.* Koppes, 180 Iowa 1268, 163 N. W., 377; Code of Iowa, 1931, Section 12309.

[60] Fredricksen *v.* Bierent, 154 Iowa 34, 134 N. W., 432; Stone *v.* Richardson, 206 Iowa 419, 218 N. W., 332.

[61] Cheshire *v.* McCoy & Henry, 205 Iowa 474, 218 N. W., 329.

good will is the most valuable result. Recommending this procedure to his client, explaining it to the client and to adjoining owners, and assisting in its consummation, is in some cases the most valuable service that a surveyor can render.

Riparian Boundaries

Another situation in which the surveyor's most important contribution may be in the nature of consultation rather than in the execution of surveys is that in which riparian boundaries are concerned.

A stream is a very natural boundary of lands. Even in the cases of smaller streams—and it is still more true of the larger ones—it is more convenient to cultivate or occupy separately the areas on opposite sides of the stream. As a boundary, a stream is easy to describe and comparatively unmistakable to find and recognize. It is visible—no surveyor is needed to interpret the description or trace the boundary. In the case of a small, non-navigable stream separating the lands of two owners there is little uncertainty about the boundary. The owners of the adjoining land own the bed of the stream; and their common boundary is generally stated to be the "thread of the stream," or the "center of the stream."[62] There is some disagreement among the Courts on how to find the thread or center of the stream,[63] but the weight of authority seems to be that the "thread of the stream" means the "center of the main channel" thereof.[64]

The definition of a navigable stream, for boundary purposes, differs materially in different jurisdictions. The seaboard States, following, as Colonies, the practice in England, adopted and have to a great extent retained the criterion that only streams in which the tide ebbs and flows are navigable. Obviously, this definition is not capable of sensible application in the interior of the country;[65] and most of the States other than those mentioned have adopted, substantially, the definition laid down by the Supreme Court of the United States, that, to be classed as navigable, a stream should in its natural state afford a channel for useful commerce.[66] It should be noted that if a stream has once had the character, for boundary purposes, of a navigable stream, and boundaries have thereby become determined, a subsequent declaration by legislative enactment that the stream is not navigable does not alter the boundaries.[67]

The Supreme Court of the United States has held that the extent of private ownership of lands bordering on navigable streams is for the respective States to determine. Each State may determine for itself whether it will keep the title to the bed of the stream, or whether it will relinquish this

[62] Moffett *v.* Brewer, 1 G. Green (Iowa), 348, 358; State *v.* Livingston, 164 Iowa 31, 145 N. W., 91.

[63] Franzini *v.* Layland, 120 Wis. 72, 97 N. W., 499; Hopkins Academy *v.* Dickinson, 9 Cush. (63 Mass.), 544, 552.

[64] Franzini *v.* Layland, *supra.*

[65] Barney *v.* Keokuk, 94 U. S. 324, 338, 24 L. Ed., 224.

[66] The Montello, 11 Wall. (78 U. S.), 411; Oklahoma *v.* Texas, 258 U. S. 574, 586, 66 L. Ed., 771, 42 Sup. Ct., 406.

[67] Park Comr's. *v.* Taylor, 133 Iowa 453, 458, 108 N. W., 927; Berry *v.* Hoogendoorn, 133 Iowa 437, 440, 108 N. W., 923.

title to the private riparian owner, retaining only such an easement for public use as that retained in the case of highways where the land on which the highway is built is under private ownership. The question, then, is one of State law.[68] In Iowa, the boundary of private ownership along a navigable stream is the high-water mark.[69] A definition of high-water mark as a boundary of private ownership laid down by the United States Supreme Court and cited approvingly by the Supreme Court of Iowa is as follows:

"The high water mark of a navigable stream, the line delimiting its bed from its banks, is to be found by ascertaining where the presence and action of water are so usual and long continued in ordinary years as to mark upon the soil of the bed a character distinct from that of the banks in respect to vegetation and the nature of the soil."[70]

The Iowa Court also states: "What the river does not occupy long enough to wrest from vegetation, so far as to destroy its value for agriculture, is not river bed."[71]

In Iowa, however, the question of navigability of a stream is not necessarily the determining factor in the matter of the location of the riparian boundary. The Iowa Court has held that not only on navigable streams, but also on non-navigable, but meandered, streams, the boundary of the riparian owner is the high-water mark:

"The action of the Land Department of the United States Government in meandering the stream and conveying the land bordering on such stream with reference to the meander line is conclusive that the stream was navigable in such sense that the title of the riparian owners resting on such survey extended, under the rule in this State, only to high-water mark."[72]

Hence, it would appear that the matter of riparian boundaries in Iowa is relatively clear. If during the original Government survey the stream was meandered, the boundary is the high-water mark; if it was not meandered, the boundary is the thread of the stream.

It is believed that the popular misconception that the meander line is a boundary is not generally shared by engineers, and it will not be discussed herein. Suffice it to say that a meander line can be a boundary only when there is no body of water corresponding to such meander line,[73] or when, in a very exceptional case, involving fraud or gross error on the part of the Government surveyor, the meander line is so very far from the high-water mark which it was supposed to approximate, that justice and reason demand that the meander line be substituted for the high-water mark as a private boundary.[74]

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[68] Oklahoma *v.* Texas, 258 U. S. 574, 595, 66 L. Ed., 771.

[69] Musser v. Hershey, 42 Iowa. 356; Barney *v.* Keokuk, 94 U. S. 324, 24 L. Ed., 224; City of Cedar Rapids *v.* Marshall, 199 Iowa 1262.

[70] Howard *v.* Ingersoll, 13 How. (54 U. S.) 381, 427, 14 L. Ed., 189.

[71] Houghton *v.* The C., D., & M. R. Co., 47 Iowa, 370, 374; Curtis *v.* Schmidt, 212 Iowa, 1279.

[72] Park Commrs. *v.* Taylor, 133 Iowa, 453, 457–8; 108 N. W., 927; Watt *v.* Robbins, 160 Iowa 587, 601, 142 N. W., 387.

[73] Carr *v.* Moore, 119 Iowa 152, 93 N. W., 52.

[74] Barringer *v.* Davis, 141 Iowa 419, 427, 120 N. W., 65, and cases there cited.

Unfortunately, while a stream is a very obvious boundary, it is not always a stable one. An extended discussion of the effect on riparian boundaries of the shifting of the stream is not appropriate to the purpose of this paper. The rule is, in general, well known, that if the shifting takes place through erosion or accretion the boundary likewise shifts with the stream.[75] If the change is made by avulsion, the boundary is not thereby moved.[76] It is sometimes extremely difficult to determine whether a given case is one of erosion or avulsion. The decisions cited discuss this distinction.

Conclusion

As in many things, so in the matter of uncertain boundaries, "an ounce of prevention is worth a pound of cure." How much better it would be if surveyors and land-owners could appreciate the importance of so marking boundaries when they are first established that there will be little likelihood of their being obliterated, and of so referencing them that they can be re-established economically and precisely if that should ever be required. By the progress of the precise surveys of the United States Coast and Geodetic Survey the most reliable and satisfactory control is being made more and more widely available. While the general use of this control for the description of the boundaries of lands is probably still in the distant future, engineers should be in the lead in recognizing its utility and its value, and in urging and facilitating the adoption of this means of increasing the certainty and permanence of private boundaries.

In the meantime, and even with the use of the control mentioned, the best and most economical assurance that a boundary can be recovered in the future in its original location is the use of definite, distinctive, and enduring monuments; and the best law that can be promulgated to contribute to this end, is that which was laid down many centuries ago: "Thou shalt not remove thy neighbor's landmark, which they of old time have set."[77]

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[75] Philadelphia Co. *v.* Stimson, 223 U. S. 605, 624, 56 L. Ed., 570; Hohl *v.* Iowa Cent. R. Co., 162 Iowa 66, 143 N. W., 850.

[76] Nebraska *v.* Iowa, 143 U. S. 359, 36 L. Ed., 186.

[77] Deut., 19:14.

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PAPERS

PHOTO-ELASTIC ANALYSIS OF STRESSES IN COMPOSITE MATERIALS

BY A. H. BEYER,[1] M. AM. SOC. C. E., AND A. G. SOLAKIAN,[2] ASSOC. M. AM. SOC. C. E.

SYNOPSIS

An analysis on models, by photo-elastic methods, of the stresses resulting from shrinkage and external loading in reinforced concrete structures, is described in this paper. The models used consist of bakelite reinforced with aluminum rods. The results that have been obtained by photo-elasticity check quite well with the accepted theory of stress analysis developed for reinforced concrete beams.

INTRODUCTION

Glass and celluloid have been used extensively for the purpose of determining the stresses in models by photo-elastic analysis. Because of its higher optical sensitivity bakelite has recently come into use. Physical tests show that this material, reinforced with aluminum, approximates the behavior of concrete, reinforced with steel.[3] At room temperatures, fortunately, the coefficient of expansion of bakelite is nearly the same as that of aluminum. The stresses in a specimen reinforced with aluminum, resulting from the application of a system of external loads, can be readily evaluated by subtracting the initial or internal stresses resulting from shrinkage and cooling, after casting, from the gross stresses developed under load. In such a specimen it is also possible to make a qualitative study of the shrinkage stresses, which approximate those developed in the reinforced concrete prototype.

A number of papers were published[4] in 1932 and 1933 containing detailed descriptions of the polariscope used, the theory and application of photo-

NOTE.—Discussion on this paper will be closed in December, 1933, *Proceedings*.

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[3] *Proceedings*, Am. Soc. C. E., September 1932, p. 1248.

[4] *Transactions*, A.S.M.E. (Applied Mechanics), June 15, 1932, pp. 97-123; *Journal*, Optical Soc. of America, May, 1932, pp. 293-306; and, *Product Engineering*, April, 1933.

elasticity to stress analysis, the properties of the test materials, etc. In addition, Coker and Filon[5] are authorities on this subject.

Fringe-Stress Equivalents

In order to evaluate the stress equivalents of the isochromatic fringes—that is, the boundary lines between the color bands which are produced when polarized white light is passed through an isotropic transparent material under stress—for the particular bakelite used in this investigation, a 0.267-in. by 1.15-in. calibrating beam of the same material, cast and cured under identical conditions as the models, was analyzed in the polariscope. This beam was subjected to pure bending moments of known intensities (38.7 in.-lb, and multiples thereof). Fig. 1 is a black and white photograph of

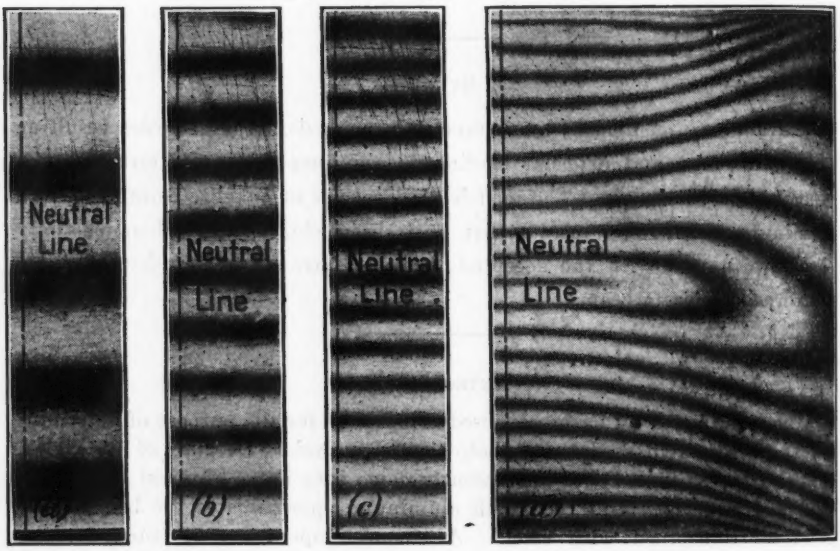

Fig. 1.—Stress Fringes in a Calibration Beam in Pure Bending: (*a*) For $M_b = 38.7$ Inch-Pounds; (*b*) for $2M_b$; (*c*) for $3M_b$; and (*d*) for $4M_b$.

the stress fringes taken with circularly polarized white light, the green bands of the image corresponding to the black fringes in the photograph and are zones of equal principal shear intensities. The average stress equivalents of the center lines of the 1st, 2d, ..., *n*th black fringes for the given thickness of the calibration beam, were found to be,

$$258, 2 \times 258 \ldots n \times 258 \text{ lb per sq. in.}$$

as determined from the computed values of the extreme fiber stresses of the stressed beam shown in Fig. 2.

Each group of the black and white fringes corresponds to a set of yellow, red, and green fringes as projected on the screen. From lines drawn on a

[5] "Photo-Elasticity," by E. G. Coker and L. N. G. Filon, 1931.

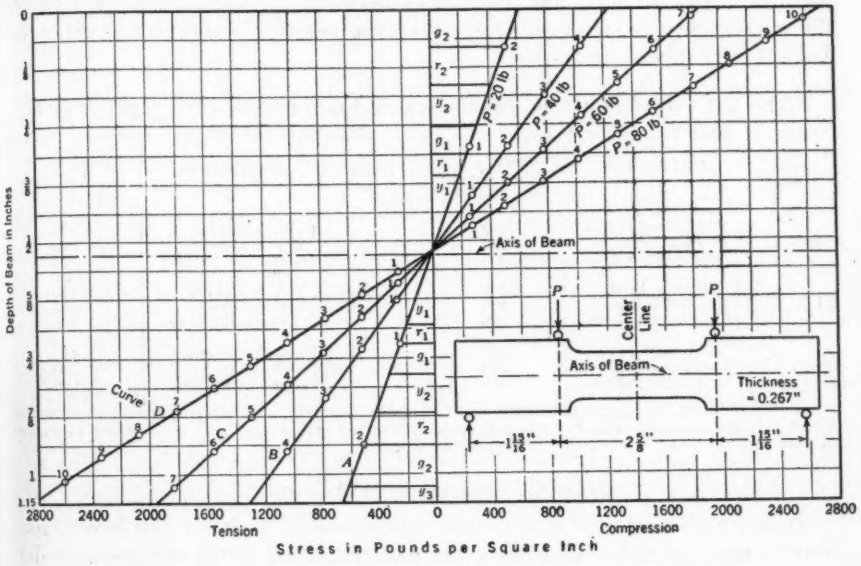

FIG. 2.—STRESS DISTRIBUTION IN CALIBRATION BEAM, IN PURE BENDING.

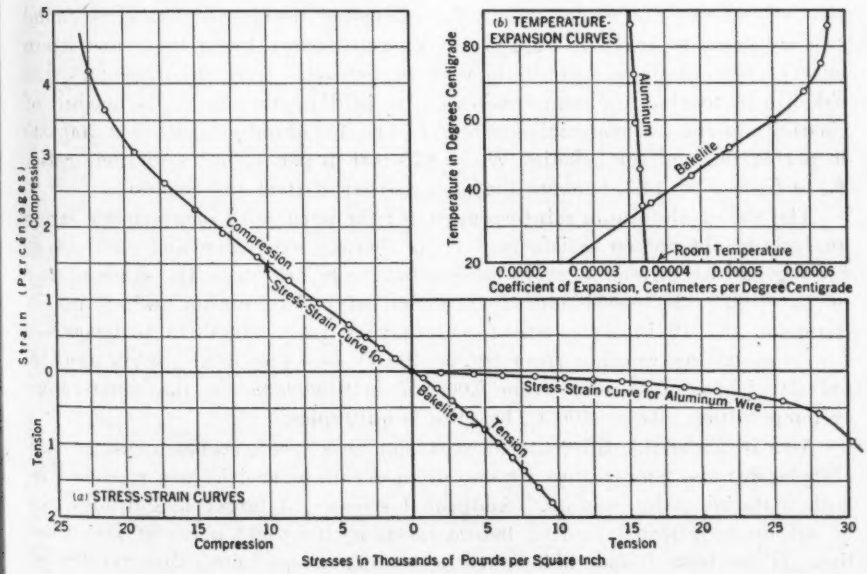

FIG. 3.—CHARACTERISTICS OF ALUMINUM AND BAKELITE: (*a*) STRESS-STRAIN CURVES; (*b*) TEMPERATURE EXPANSION CURVES.

sheet of paper marking the boundaries between the different color fringes, the average fringe-stress equivalents of the several isochromatics given in Table 1 were determined.

TABLE 1.—ISOCHROMATIC FRINGE-STRESS EQUIVALENTS (t = 0.267 INCH).

Order of color	Dividing line between:	Stress, $p-q$,* in pounds per square inch	Order of color	Dividing line between:	Stress, $p-q$,* in pounds per square inch
I.....	Yellow and red.....	185	IV.....	Yellow and red.....	875
	Red and green.....	230		Red and green.....	1 005
	Green and yellow...	305		Green and yellow...	1 110
II.....	Yellow and red.....	390	V.....	Yellow and red.....	1 150
	Red and green.....	475		Red and green.....	1 255
	Green and yellow...	575		Green and yellow...	1 365
III.....	Yellow and red.....	630	VI.....	Yellow and red.....	1 450
	Red and green.....	710		Red and green.....	1 530
	Green and yellow...	845		Green and yellow...	1 690

* p is the maximum and q the minimum principal stress and $p - q$ is equal to twice the maximum shear stress; in pure bending, $q = 0$.

BEAM REINFORCED WITH ALUMINUM

In order to be able to study stresses produced in a simple reinforced concrete element by means of models, the materials used in the composite model should have elastic properties approximating those of the reinforced concrete. Concrete and bakelite are both brittle materials. Structural steel and aluminum are both ductile. The ratio of the modulus of elasticity of steel, E_s, to that of concrete in compression, E_c, commonly designated by n, is found to vary from 10 to 15 in reinforced concrete tests. From the stress-strain curves (Fig. 3(a)) of aluminum wire in tension ($\frac{1}{16}$ in. in diameter) and bakelite in tension and compression ($\frac{3}{4}$ in. in diameter) tests, the moduli of elasticity of the two materials are found to be, for aluminum, E_a = 9 700 000 lb per sq in., and for bakelite, E_b = 670 000 lb per sq in.; and their ratio, E_a to $E_b = n = 14.5$, is about the same as that of steel and concrete.

The use of aluminum reinforcement for the purpose of photo-elastic stress analysis would not be satisfactory if its thermal expansion and contraction for thermal conditions at room temperature were not about the same as that of bakelite. The coefficient of expansion of an aluminum rod, $\frac{3}{8}$ in. in diameter and 12 in. long, was found to vary from 0.0000375 to 0.0000355 for temperatures ranging from 20° to 100°C (see Fig. 3(b)), while that of bakelite is found to vary from 0.000027 to 0.0000625 for the same range of temperature. Above 100°C, bakelite is quite plastic.

As a result of the difference of expansion in a specimen beam cast around aluminum rods, temperature stresses of moderate intensities are produced in both materials after cooling. Additional stresses of larger magnitudes can be set up by external loading before reaching the yield point of the bakelite. It has been found that careful annealing of specimens thus reinforced reduces the intensities of the shrinkage stresses.

Fig. 4(a) is a view of a fringe pattern of the initial stresses in a test beam reinforced with a piece of aluminum wire having its two ends bent at

an angle. After applying a pure bending moment to the beam, the new fringe pattern, Fig. 4(*b*), appears. A complete stress analysis (with a view of the fringe pattern) of a plate reinforced with a straight bar of aluminum has been published by the writers in a discussion of another paper.[6] Fig. 13 of that discussion shows heavy stress concentrations near the ends of the rod due to shrinkage of the beam during the cooling process.

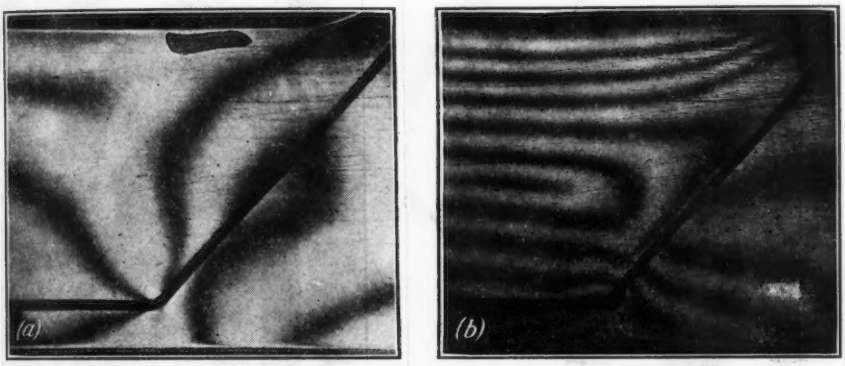

FIG. 4.—TEST BEAM REINFORCED WITH INCLINED ROD: (*a*) INITIAL STRESSES ONLY; (*b*) INITIAL STRESSES PLUS BENDING.

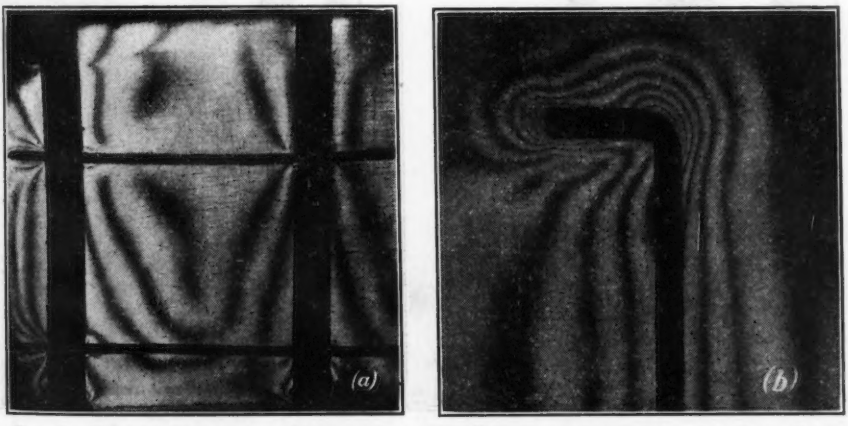

FIG. 5.—STRESS ANALYSIS OF HOOKED ROD, AND FLAT SLAB ANALYSIS: (*a*) TEST SLAB REINFORCED WITH CROSS-RODS; (*b*) ALUMINUM ROD BENT 90 DEGREES.

Fig. 5(*a*), a fringe pattern for a slab reinforced with longitudinal and transverse bars, shows the absence of material shrinkage stresses in the enclosed square, as compared to those along the edges. This is due to the beneficial effect of the transverse reinforcing bars. Fig. 5(*b*) shows the internal stress configuration adjacent to a 90° bend of a rod anchored in a plate. Note the high shear stress concentrations at the convex side of the sharp radius bend.

[6] *Proceedings*, Am. Soc. C. E., September, 1932, p. 1251.

Fig. 6(*a*) is a fringe pattern of the internal shear stresses developed in a plate similar to Fig. 5(*b*), reinforced with a rod having a large-radius bend or hook at the anchored end. On the outside of the bend the stress fringes are almost parallel to the curvature of the rod, indicating a uniform dis-

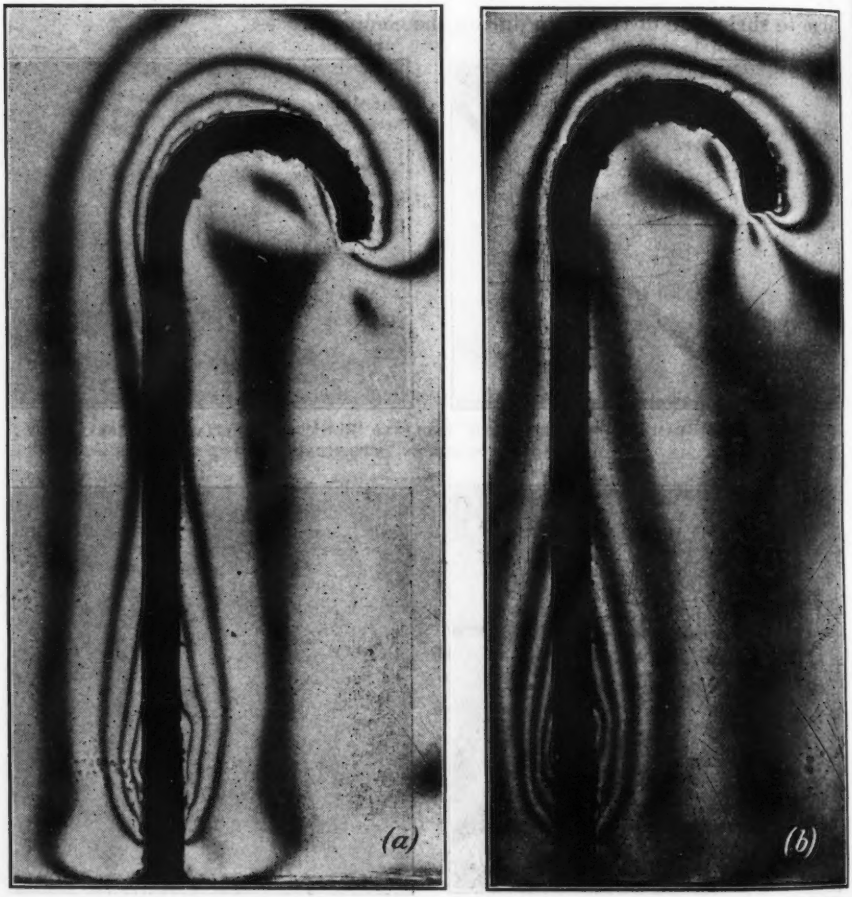

FIG. 6.—STRESS FRINGES AROUND SEMI-CIRCULAR HEAD OF ROD EMBEDDED IN PLATE: (*a*) INITIAL STRESSES; (*b*) INITIAL STRESSES PLUS PULL OF 100 POUNDS ON ROD.

tribution of the bond stresses along that section of the rod, as compared to high stress concentration at the corresponding region of the right-angled sharp bend. After applying a pull of 100 lb to the lower end of the rod in Fig. 6(*a*), the fringe pattern obtained is as shown in Fig. 6(*b*). The variation of the maximum shear stresses along the inner and outer boundaries of the rod are given in Fig. 7, which shows graphically the initial stresses, the combined stresses resulting from the 100-lb tensile load and the initial stresses, and, finally, the net stresses due to the load alone. In all cases

the highest shear stresses are found a short distance in from where the rod leaves the plate. This phenomenon may account for the cracks and crumbling off of concrete at the point where a reinforcing bar projects from a concrete mass.

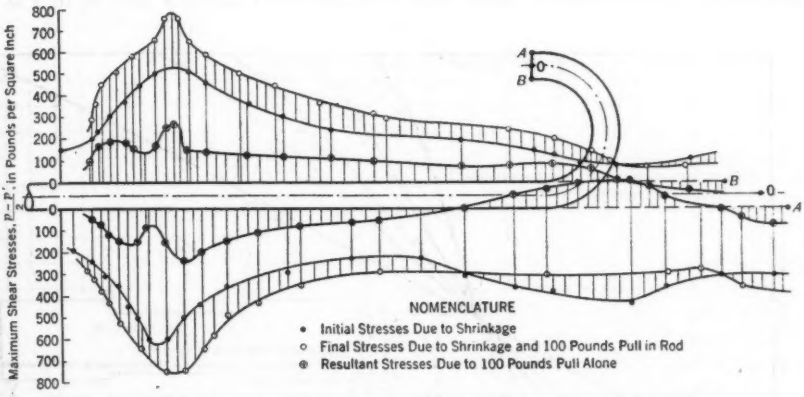

FIG. 7.—DISTRIBUTION OF MAXIMUM SHEAR ALONG THE BOUNDARIES OF A REINFORCEMENT ROD.

ANALYSIS OF A BEAM REINFORCED WITH A STRAIGHT ROD

The specimen, a beam 0.266 in. wide and 1.15 in. deep, was machined out of a bakelite casting (1 in. by 2.5 in. in cross-section). The beam had been reinforced with a rod, 0.05 in. in diameter, placed at a distance of 1 in. from the upper extreme fiber of the beam. After being polished and subsequently annealed, the specimen was analyzed for shrinkage stresses and also under four different intensities of pure bending. Fig. 8 shows, in black and white,

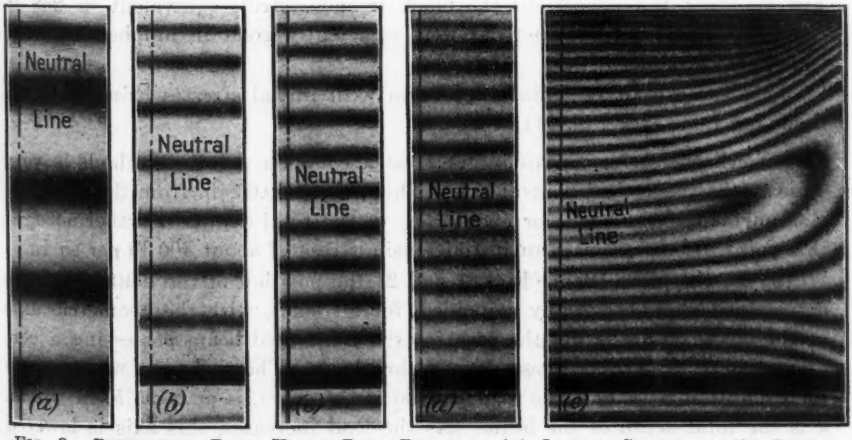

FIG. 8.—REINFORCED BEAM UNDER PURE BENDING: (*a*) INITIAL STRESSES; (*b*) INITIAL STRESSES AND $M_b = 35$ INCH-POUNDS; (*c*) INITIAL STRESSES AND $2M_b$; (*d*) INITIAL STRESSES AND $3M_b$; AND (*e*) INITIAL STRESSES AND $5M_b$.

the stress fringes for the unloaded and loaded conditions. The curves in Fig. 9 show graphically the variation of these stresses over the transverse mid-section. On account of the small number of black fringes present for test conditions, $P = 0$ and $P = 20$ lb, the isochromatic fringe lines (stress equivalents from Table 1) were used to determine Curves A and B, of Fig. 9.

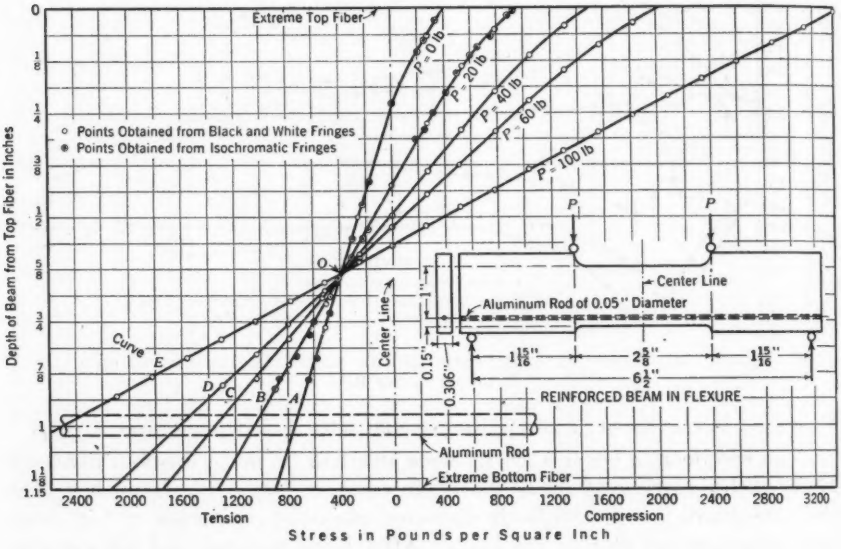

FIG. 9.—STRESS DISTRIBUTION IN A REINFORCED BEAM DUE TO INITIAL STRESS AND PURE BENDING.

For all other test conditions the black fringes (stress equivalent = 258 lb per sq in. and multiples thereof) were sufficiently great in number to locate Curves C, D, and F.

This analysis of the reinforced beam with initial stresses brings out the following facts (see Fig. 9):

(*a*) All the stress curves intersect at a common point, O, which is very close to the theoretical neutral axis. This point is 0.63 in. from the compression surface of the beam, or about 55% of the total depth. Furthermore, at this point, O, there exists an initial tensile stress of about 400 lb per sq in.

(*b*) In the plain beam (Figs. 1 and 2) the position of the neutral axis as determined photo-elastically coincided, for all loads, with the geometric axis of the beam. This is not the case for the reinforced beam possessing a certain amount of initial stress due to shrinkage. The apparent neutral axis for Condition A, Fig. 9 (no load; initial stress only), is at 0.195 h, in which, h is the total depth of the beam. As the load increases, this axis is lowered and tends to approach the true neutral axis as determined by theory. This is shown clearly in Figs. 8 and 10.

(*c*) When the internal shrinkage stresses are included the compressive stresses are not directly proportional to their distances from the neutral axis of the beam.

This phenomenon of a shifting neutral axis and a non-linear stress variation, as outlined under Facts (*b*) and (*c*), does not affect the validity of the assumptions made in the common theory of flexure of non-homogeneous

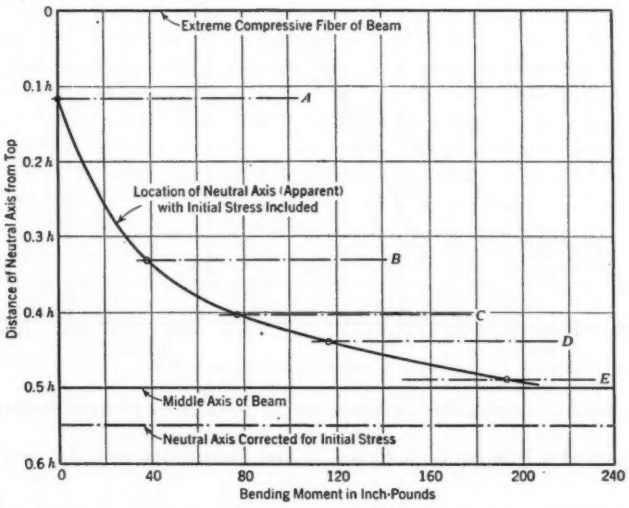

FIG. 10.—LOCATION OF NEUTRAL AXIS IN A REINFORCED BEAM, WITH INITIAL STRESSES.

materials under bending stresses. This fact is illustrated by Fig. 11 which gives the net stresses resulting from external loading alone. These stresses were obtained from the curves in Fig. 9 by subtracting the shrinkage stresses from the combined shrinkage and bending stresses. The position of the true neutral axis for the net stresses resulting only from the applied loading, as determined photo-elastically, is independent of the intensity of the loading. For the test specimen used, this axis is about 0.630 in. from the upper compressive face of the beam, 0.55 h, approximately at the common point of intersection of the curves of gross stresses (Fig. 9). The computed location of the neutral axis of this beam is 0.601 in. from the top surface, which checks closely with that determined photo-elastically. (Contrary to the common practice in the design of reinforced concrete structures, of neglecting the tensile strength of concrete in computations, the bakelite is assumed to take its full share in tension in this analysis.)

It is interesting to observe that in pure bending the stresses due to loading alone are, in reinforced bakelite, directly proportional to the intensity of the bending moments. The common theory of flexure developed for non-homogeneous beams appears to apply. Using this theory and the group of curves designated by B', C', D', and E' (Fig. 11) for net stresses, the values of

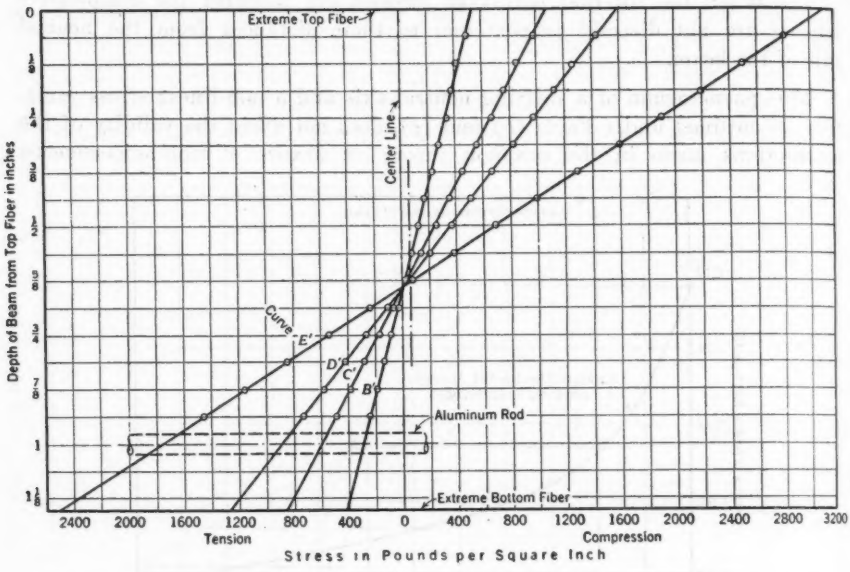

FIG. 11.—STRESS DISTRIBUTION IN A REINFORCED BEAM, DUE TO PURE BENDING ONLY.

$n = \frac{E_a}{E_b}$, and the intensity of stress in the aluminum rod, f_a, can be determined as follows:

In Fig. 12, let $t = 0.266$ in.; $h = 1.150$ in.; $h_1 = 0.767$ in.; $h_2 = 0.800$ in.; $x = 0.601$ in.; and, $n = \frac{E_a}{E_b} = 14.5$. Consider a part of the reinforced beam under a uniform external bending moment, M_b, to be in equilibrium by the internal resisting moment, M_r, of the stresses developed in the composite materials. For equilibrium conditions, therefore, $\Sigma H = 0$, or,

$$T_a + T_b = C \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

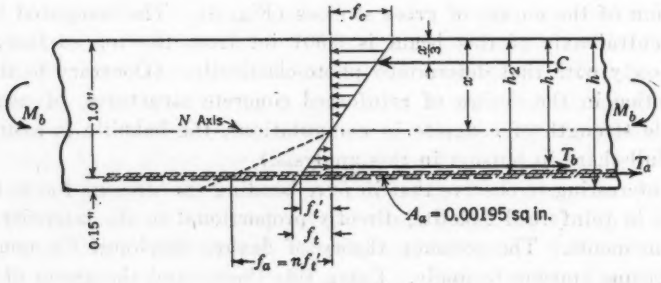

FIG. 12.

Furthermore, $\Sigma M = 0$, or,

$$M_b = M_r = T_b h_1 + T_a h_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The total tension in bakelite is,

$$T_b = \frac{f_t}{2} [(h - x)\, t - A_a] \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The total tension in aluminum is,

$$T_a = f_a A_a = n\, f'_t\, A_a \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The total compression in the beam is,

$$C = T_a + T_b = \frac{f_c}{2}\, x\, t \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

and from Equation (4), the stress in aluminum is,

$$f_a = \frac{T_a}{A_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Values of f_a for different loading conditions may be computed by Equation (6) and are given in Table 2. The fiber stresses, f_t, f'_t, and f_c in Columns

TABLE 2.—FLEXURE TEST OF A BEAM REINFORCED WITH A STRAIGHT ROD ($h_1 = 0.767$ in.; $h_2 = 0.79$ in.; $h = 1.15$ in.; $x = 0.63$ in.; $t = 0.266$ in; and $A_a = 0.00196$ sq in.)

Curve (Fig. 11)	Applied bending moment, M_b, in inch-pounds	Unit Stress, in Pounds per Square Inch			Total Stress, in Pounds			Unit stress, f_a, in reinforcing bar (Equation (6))	Resisting moment, M_r, in inch-pounds (Equation (2))	Percentage error, $\frac{M_r - M_b}{M_r} \times 100$
		Maximum tensile stress, f_t, in bakelite (Fig. 11)	Tensile stress in bakelite, f'_t, in the plane of the axis of the reinforcing rod (Fig. 11)	Maximum compressive stress, f_c, in bakelite (Fig. 11)	Tension in bakelite (Equation (3))	Compression in bakelite, C (Equation (5))	Tension in aluminum, T_a (Equation (4))			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
B'.....	35	455	335	500	31.0	41.9	10.9	5 590	32.4	— 8.0
C'.....	70	880	635	1 030	60.0	186.3	26.3	13 480	66.8	— 4.6
D'.....	105	1 290	925	1 540	88.0	129.1	41.1	21 070	100.0	— 5.0
E'.....	175	2 520	1 810	3 060	171.8	256.4	84.6	43 380	198.6	+11.8

(3), (4), and (5), Table 2, are corrected by 45 lb per sq in. because the common point of intersection of the net-stress curves in Fig. 11 are on the compression side of the zero stress line by that amount. If values in Columns (2), (3), (5), and (9), Table 2, are plotted against values of M_b in Column (2), the relation between these factors is illustrated.

CONCLUSIONS

The photo-elastic analysis made upon the test beam described in this paper, reinforced in tension with a straight rod of aluminum and subject to pure bending, gave the following results:

(1) The stress analysis made by the photo-elastic method substantially satisfies the static equilibrium condition (see Columns (2) and (11), Table 2).

(2) The stresses in both aluminum and bakelite were found to be substantially proportional to the external bending moment for all stresses less than about 20% of the ultimate tensile strength of the bakelite.

(3) The theoretical location of the neutral axis checks closely with that determined by photo-elastic methods, but only after deducting the initial, or so-called internal, stresses.

(4) By means of photo-elastic methods the initial stresses in composite materials, and the effect of these initial stresses when superimposed upon the loading stresses, can be determined qualitatively, all of which results, in a pronounced shifting of the neutral axis.

Acknowledgment

The writers are greatly indebted to Messrs. T. S. Taylor and J. H. Schmidt, of the Bloomfield (N.J.) Research Laboratories of the Bakelite Corporation, for their kind co-operation in preparing the reinforced specimens analyzed in this paper.

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PAPERS

WATER-BEARING MEMBERS OF ARTICULATED BUTTRESS DAMS

BY HAKAN D. BIRKE,[1] JUN. AM. SOC. C. E.

SYNOPSIS

This paper presents a scientific method of determining mathematically, the most economical proportions of the water-bearing members of the articulated type of buttress dam.

INTRODUCTION

Wide buttress spacings combined with massive masonry sections are improvements in buttress dam design that have accompanied the recent trend toward the use of dams of great height. Greater safety and economy have been attained through these improvements, and through the more scientific methods of design that have been developed at the same time. At first, the use of a wide buttress spacing was confined to the multiple-arch and multiple-dome types of dam. However, the unsuitability of these types to faulted, or otherwise poor, foundations has led to the improvements of the well-known flexible, "articulated," type of dam, using thick, widely spaced buttresses instead of the relatively thin, closely spaced buttresses that have been common to this type. Fortunately, the characteristics of massiveness and wide spacing have been found to be as economical for articulated dams as they were for the rigid types and, in addition, the flexibility of this type makes it suitable for sites with poor foundations, such as are now being frequently encountered, and for those subjected to earthquake shocks or other foundation movements.

The use of thick buttresses and wide spacing has introduced new problems in the design of the water-bearing members or decks and the haunches. Two types of flexible water-bearing members are suitable for use in connection with long spans, the first of which (Fig. 1(*b*)), consists of a gradually flaring buttress section connected with a short reinforced concrete slab[2];

NOTE.—Discussion on this paper will be closed in December, 1933, *Proceedings*.

[1] Designing Engr., Ambursen Dam Co., New York, N. Y. Elected an Associate Member, Am. Soc. C. E., on July 25, 1932.

[2] "Recent Advances in Buttress Type Dams," by Calvin V. Davis, M. Am. Soc. C. E., *Civil Engineering*, February, 1931.

and the other (Fig. 1(*c*)), is the round-head type.[3] For purposes of comparison a standard Ambursen type deck is shown in Fig. 1(*a*). It will be noted (Fig. 1(*b*)) that for the dimension, $b = 0$, this type will be identical with the original type shown in Fig. 1(*a*), and for $b = L_2$ (see Fig. 1(*b*)), the deck will be replaced entirely by the enlarged haunches shown in (Fig. 1(*c*)).

Heretofore, the economic limits of each of the three types shown in Fig. 1 have been determined by the tedious method of trial designs. It is a matter of considerable economic importance to determine the correct relations be-

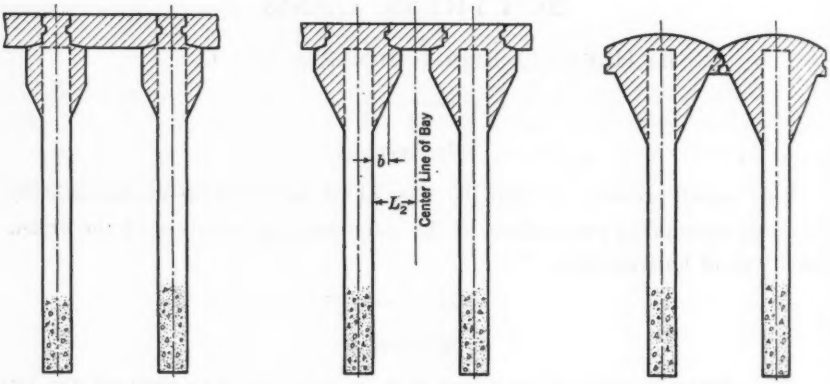

FIG. 1.

tween length of span, bearing surface of slab, depth of haunch, slope of haunch, the height at which the deck slab may be omitted, and the height at which the flared buttress sections may be brought in contact to form a round-head type of water-bearing member. In order to facilitate the economic design of these flexible water-bearing members, equations and curves have been developed, through which the relations outlined in the foregoing are easily and quickly obtained.

NOTATION

The following symbols adopted for use in this paper, are listed for the guidance of discussers:

a = width of haunch bearing (Fig. 2).

b = one-half the difference between tongue and buttress thickness (Fig. 1(*b*)).

c = one-half the buttress thickness (Fig. 4(*b*)).

d = effective depth: d_1, at the outer face; and d_2, the depth of deck (Fig. 4(*b*)).

e = (a ratio, multiplied by 100) the increase in percentage of steel due to bent-up bars and hooks: e_1, haunch steel; and e_2, deck steel (Equation (34)).

[3] "The Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E.

f = allowable unit stress: f_c, in concrete; and f_s, in steel (Equation (35)).
i = depth of cover for steel: i_1, in haunch; and i_2, in deck (Equation (18)).
j = factor of distribution of shear: j_1, in haunch; and j_2, in deck.
p = unit loading: p_0, maximum on haunch, from the deck (Equation (1)).
s = tangent of the slope of the under side of the haunch on sections normal to the deck (Fig. 2).
x = variable distance along an X-axis.
A = substitution factor, with appropriate subscript (Equation (8)).
B = substitution factor, with appropriate subscript (Equation (25)).
C = total cost, in dollars: C_c, of concrete in deck and haunch; C_h, of steel in haunch; C_d, of steel in deck; C_t, of concrete and steel in deck and haunch.
D = unit cost: D_c, of concrete, in dollars per unit volume; D'_c, of concrete, in dollars per cubic yard; D_s, of steel, in dollars per unit volume; D'_s, of steel, in dollars per pound.
E = modulus of elasticity.
F = substitution factor, with appropriate subscript.
G = substitution factor, with appropriate subscript.
H = depth below maximum high-water level to the under side of the deck.
J = substitution factor, with appropriate subscript.
K = substitution factor, with appropriate subscript.
L = one-half span: L_1 = one-half the clear deck span; and L_2, one-half the clear buttress spacing.
M = bending moment.
N = substitution factor, with appropriate subscript.
Q = substitution factor, with appropriate subscript.
R = area of steel: R_1 = area required for a given bending moment.
V = volume: V_c, of concrete in deck and haunch; V_h, of steel in haunch; and V_d, of steel in deck.
W = loading on deck per unit area.
τ = allowable shear in haunch.
μ = coefficient of friction.
v = allowable shear in deck; $v_{max.}$ = maximum shear.
σ = principal stress: σ_1 equals first principal stress in buttress.

The symbols for costs and volumes of materials in haunch and deck are given for one-half a bay, and per unit length of deck and haunch (see Fig. 4).

Relation of Haunch Dimensions to Loading Distribution and Haunch Slope

The haunch may be considered as a cantilever beam with length, a (see Fig. 2). The loading from the deck is assumed to be triangularly distributed with a maximum value of p_0. The effective depth of the haunch at the outside face is equal to d_1 and the cover for the steel is equal to i_1. As the length of this beam will be small compared with the depth, the maximum allowable shear will govern the dimensions. The location of the section in the haunch where the shear is maximum must be determined, and the haunch

designed considering the shear at this section. For example, consider the part of the haunch between the plane, *A-A* (Fig. 2), and the outer face of the haunch.

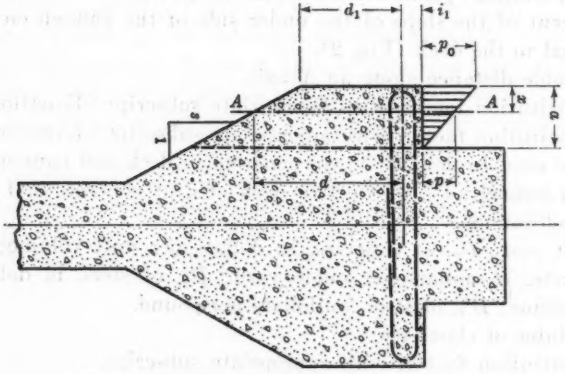

FIG. 2.—NORMAL SECTION THROUGH HAUNCH.

Let τ = shear along the plane, *A-A*; p = unit loading at the plane, *A-A*; and d = effective depth of haunch at the plane, *A-A*. Then,

$$p = p_o - \frac{x}{a} p_o \quad \ldots\ldots (1)$$

and,

$$d = d_1 + sx \quad \ldots\ldots (2)$$

Summing up the vertical forces for the element of haunch limited by the plane, *A-A*, will give Equation (3):

$$\tfrac{1}{2} (p + p_o) x - j_1 \tau d = 0 \quad \ldots\ldots (3)$$

Combining Equations (1), (2), and (3) and re-arranging,

$$j_1 \tau d_1 = x (p_o - \tau j_1 s) - \tfrac{1}{2} p_o \frac{x^2}{a} \quad \ldots\ldots (4)$$

The distance, x, should be determined so that the depth, d_1, becomes a maximum. Therefore,

$$\frac{\partial d_1}{\partial x} = 0 \quad \ldots\ldots (5)$$

From Equations (4) and (5), by differentiating and solving for x,

$$x = \left(1 - \frac{j_1 \tau}{s}\right) a \quad \ldots\ldots (6)$$

Substituting this value of x in Equation (4) and solving for d_1 will give,

$$d_1 = \tfrac{1}{2} \frac{j_1 \tau}{p_o} \left(\frac{p_o}{j_1 \tau} - s\right)^2 a = Aa \quad \ldots\ldots (7)$$

Cancelling a in Equation (7):

$$2 \frac{p_o}{j_1 \tau} A = \left(\frac{p_o}{j_1 \tau} - s\right)^2 \quad \ldots\ldots (8)$$

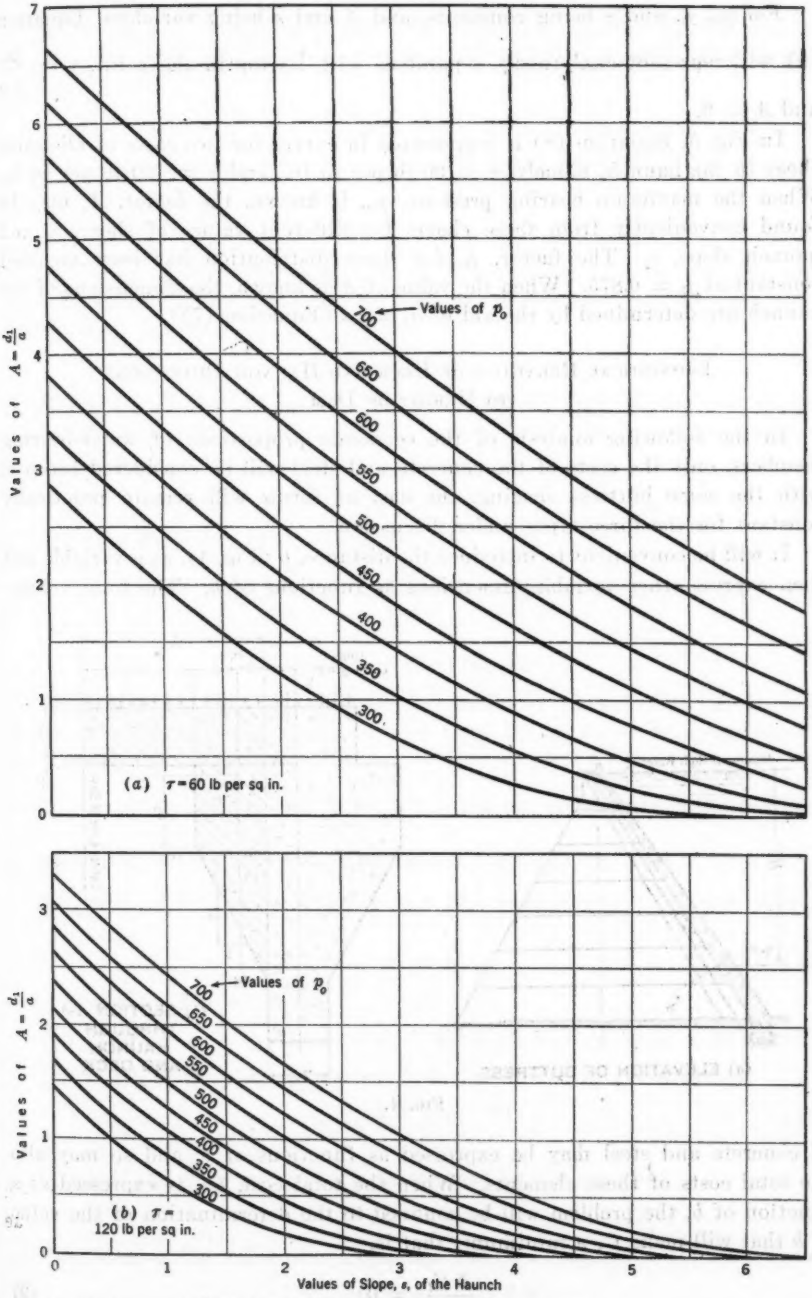

FIG. 3.—VALUES OF A IN EQUATION (8).

For p_o, j_1, and τ being constants, and A and s being variables, Equation (8) will represent analytically a parabola with its top in the point, $s = \frac{p_o}{j_1 \tau}$ and $A = 0$.

In Fig. 3, Equation (8) is represented in curves for two cases of allowable shear in the haunch, namely, $\tau = 60$ lb per sq in., and $\tau = 120$ lb per sq in. When the maximum bearing pressure, p_o, is known, the factor, A, may be found conveniently from these charts for different values of shear, τ, and haunch slope, s. The factor, j_1, for shear distribution has been assumed constant at $j_1 = 0.875$. When the value of A is known, the dimensions of the haunch are determined by the aid of $d_1 = Aa$ (Equation (7)).

Economical Relations of Deck and Haunch Dimensions to Height of Dam

In the following analysis of the economic proportions of water-bearing members, only the costs of the concrete and steel will be considered because, with the same buttress spacing, the cost of forms will remain practically constant for the three types under discussion.

It will be convenient to introduce the distance, b (Fig. 4), as a variable and then express other variable dimensions as functions of b. The total volume

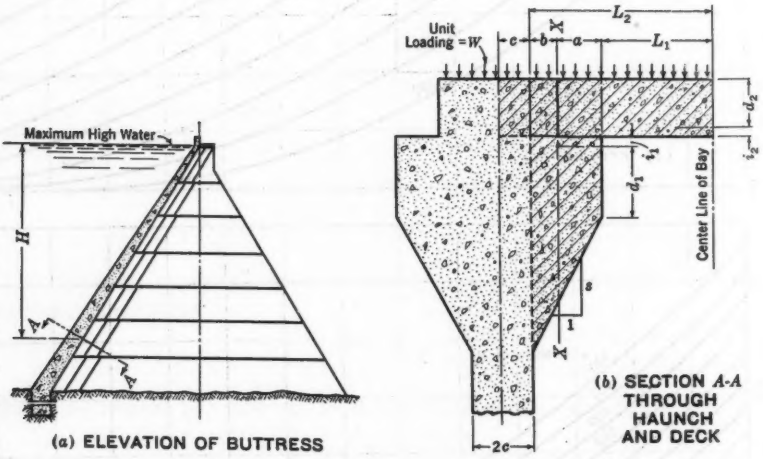

FIG. 4.

of concrete and steel may be expressed as functions of b, and so may also the total costs of these elements. When the total cost, C_t, is expressed as a function of b, the problem will be confined to the determination of the value of b that will make C_t a minimum; that is,

$$\frac{\partial C_t}{\partial b} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Let V_c = the volume of concrete in deck and haunch; D_c, the cost of concrete per unit volume; and C_c the total cost of concrete in deck and haunch. Then,

$$C_c = D_c V_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

In the same manner, the cost of the steel in the haunch will be expressed by the equation,

$$C_h = D_s V_h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

in which, V_h = the volume of steel in the haunch; D_s, the cost per unit volume of steel; and C_h, the cost of steel in the haunch. For the deck steel, the equation will be:

$$C_d = D_s V_d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

in which, V_d = the volume of steel in the deck; and C_d, the cost of the steel in the deck. The total cost, C_t, of concrete and steel in haunch and deck will be determined by the equation,

$$C_t = C_c + C_h + C_d = D_c V_c + D_s (V_h + V_d) \quad \ldots\ldots\ldots\ldots (13)$$

Applying Equation (9) to Equation (13),

$$D_c \frac{\partial V_c}{\partial b} + D_s \left(\frac{\partial V_h}{\partial b} + \frac{\partial V_d}{\partial b} \right) = 0 \quad \ldots\ldots\ldots\ldots\ldots (14)$$

Solution of Equation (14)

There will be two general cases of analysis: Case I, in which shear governs the thickness of the deck; and Case II, in which bending governs the thickness of the deck.

Case I: Determination of $\frac{\partial V_c}{\partial b}$.—The volume of concrete in haunch and deck per unit of length (see hatched area in Fig. 4(*b*)) = V_c:

$$V_c = (L_2 + c)\, i_2 + (L_2 + c)\, d_2 + (a + b)\, i_1 + (a + b)\, d_1 + \tfrac{1}{2} (a + b)^2 s \ldots (15)$$

Let,

$$i_1 + d_1 = K_1 d_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

then, Equation (15) becomes,

$$V_c = (L_2 + c)\, i_2 + (L_2 + c)\, d_2 + (a + b)\, K_1 d_1 + \tfrac{1}{2} (a + b)^2 s \ldots (17)$$

The width of the haunch bearing, a, is determined by the equation,

$$a = \frac{2\,W\,(L_2 - b)}{p_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

One-half the clear deck span is determined by the equation,

$$L_1 = L_2 - (a + b) \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

Substituting the value of a (Equation (18)), in Equation (19),

$$L_1 = \frac{(L_2 - b)(p_o - 2\,W)}{p_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

The effective depth, d_2, of the deck as required for maximum shear, v, is determined by the usual equation,

$$d_2 = \frac{W L_1}{v j_2} = \frac{W}{v j_2} \frac{(L_2 - b)(p_o - 2W)}{p_o} \quad \ldots\ldots\ldots\ldots (21)$$

The effective depth, d_1, of the haunch is determined by Equation (7), thus $d_1 = Aa$. Substituting the value of a (Equation (18)), in Equation (7) will give, as a result, the equation,

$$d_1 = A \frac{2W(L_2 - b)}{p_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Combining Equations (18), (20), (21), and (22) with Equation (17), and re-arranging,

$$V_c = \left(1 + \frac{c}{L_2}\right) i_2 L_2 + (p_o - 2W) \frac{W}{p_o^2} \left[B + G \frac{b}{L_2} - \tfrac{1}{2} J \frac{b^2}{L_2^2}\right] L_2^2 \ldots (23)$$

in which,

$$B = \frac{4 K_1 A + 2s}{p_o - 2W} W + \frac{p_o}{v_{max.} j_2} \left(1 + \frac{c}{L_2}\right) \quad \ldots\ldots\ldots (24)$$

$$G = 2 K_1 A \frac{p_o - 4W}{p_o - 2W} + 2s - \frac{1 + \frac{c}{L_2}}{j_2} \frac{p_o}{v_{max.}} \quad \ldots\ldots\ldots (25)$$

and,

$$J = 4 K_1 A - s \left(\frac{p_o}{W} - 2\right) \quad \ldots\ldots\ldots\ldots\ldots (26)$$

Differentiating V_c with respect to b will result in the equation,

$$\frac{\partial V_c}{\partial b} = (p_o - 2W) \frac{W}{p_o^2} \left[G - J \frac{b}{L_2}\right] L_2 \quad \ldots\ldots\ldots\ldots (27)$$

Case I: Determination of $\frac{\partial V_h}{\partial b}$.—Let M equal the bending moment in the haunch at Section X-X (Fig. 4(b)), and d, the effective depth of the haunch at Section X-X; then,

$$M = (L_2 - b) W \frac{2a}{3} \quad \ldots\ldots\ldots\ldots\ldots (28)$$

and,

$$d = d_1 + a s \quad \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

or, by substituting the value for d_1 from Equation (7), $d = a(A + s)$.

Let R_1 equal the steel area required for the bending moment,

$$R_1 = \frac{2}{3} \frac{(L_2 - b) W}{(A + s) j_1 f_s} \quad \ldots\ldots\ldots\ldots\ldots (30)$$

in which, j_1 is the distribution factor for shear, and f_s, the allowable tensile stress in steel. (For other symbols, see Fig. 4.)

For movements in the deck, caused by temperature changes, etc., a pull will be exerted on the haunch due to friction between the deck and the

haunch surfaces. If μ is the coefficient of friction between deck and haunch and if R_2 is the steel area required for this pull, the following equation results for the steel area required,

$$R_2 = \frac{\mu\,(L_2 - b)\,W}{f_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

Let R_h equal the total steel area required in the haunch for bending moment and pull from the deck. Then,

$$R_h = R_1 + R_2 = \frac{L_2 - b}{f_s}\,W\left[\frac{2}{3\,j_1\,(A+s)} + \mu\right] \ldots\ldots\ldots (32)$$

Now the volume of the haunch steel is determined by the equation,

$$V_h = (1 + e_1)\,(a + b + c)\,\frac{L_2 - b}{f_s}\,W\left[\frac{2}{3\,j_1\,(A+s)} + \mu\right] \ldots\ldots (33)$$

in which, e_1 is a factor for the increase in steel due to hooks, shear bars, etc., in the haunch. Let,

$$N = (1 + e_1)\,\frac{p_o}{f_s}\left[\frac{2}{3\,j_1\,(A+s)} + \mu\right] \ldots\ldots\ldots\ldots\ldots (34)$$

Introducing the value of a in Equation (33), it may be written,

$$V_h = (p_o - 2\,W)\,\frac{W}{p^2{}_o}\left[\frac{p_o\,\frac{c}{L_2} + 2\,W}{p_o - 2\,W} + \frac{p_o\left(1 - \frac{c}{L_2}\right) - 4\,W}{p_o - 2\,W}\,\frac{b}{L_2} - \frac{b^2}{L^2{}_2}\right] N\,L^2{}_2 \ldots (35)$$

Differentiating V_h with respect to b:

$$\frac{\partial\,V_h}{\partial\,b} = (p_o - 2\,W)\,\frac{W}{p^2{}_o}\left[\frac{p_o\left(1 - \frac{c}{L_2}\right) - 4\,W}{p_o - 2\,W} - 2\,\frac{b}{L_2}\right] N\,L_2 \ldots\ldots (36)$$

Case I: Determination of $\frac{\partial\,V_d}{\partial\,b}$.—The bending moment in the center of the deck span is expressed by,

$$M = \frac{1}{8}\,W\,[2\,(L_2 - b) - a]^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (37)$$

and the steel area required for this bending moment is,

$$R_d = \frac{W\,[2\,(L_2 - b) - a]^2}{8\,j_2\,d_2\,f_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots (38)$$

Introducing values for a and d_2 from Equations (18) and (21), and simplifying:

$$R_d = \frac{1}{2\,f_s}\,\frac{\nu}{p_o}\,\frac{(p_o - W)^2}{p_o - 2\,W}\,(L_2 - b) \ldots\ldots\ldots\ldots\ldots\ldots (39)$$

Let V_d equal the volume of steel in the deck slab; then,

$$V_d = \frac{1}{2}\,(p_o - 2\,W)\,\frac{W}{p_o^2}\,F\,(L_2 - b)^2 \ldots\ldots\ldots\ldots\ldots\ldots (40)$$

in which,

$$F = (1 + e_2)\,\frac{\nu}{f_s}\,\frac{p_o}{W}\left(\frac{p_o - W}{p_o - 2\,W}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (41)$$

and e_2 is the factor for increase in steel due to hooks and bent-up bars in the deck slab.

Differentiating Equation (40) with respect to b:

$$\frac{\partial\, V_d}{\partial\, b} = -\,(p_o - 2\,W)\,\frac{W}{p_o^2}\,F\left(1 - \frac{b}{L_2}\right)L_2 \quad \ldots\ldots\ldots\ldots (42)$$

Introducing the values of $\frac{\partial\, V_c}{\partial\, b}$, $\frac{\partial\, V_h}{\partial\, b}$, and $\frac{\partial\, V_d}{\partial\, b}$ from Equations (27), (36)' and (42) into Equation (14), will result in the following equation:

$$(p_o - 2\,W)\,\frac{W}{p_o^2}\left[D_c\left(G - J\,\frac{b}{L_2}\right) + D_s\left(N\,\frac{p_o\left(1 - \frac{c}{L_2}\right) - 4\,W}{p_o - 2\,W} - 2\,N\,\frac{b}{L_2} + F\,\frac{b}{L_2} - F\right)\right]L_2 = 0 \quad \ldots\ldots\ldots\ldots (43)$$

From the expression within the square brackets, b, may be solved, thus:

$$b = \frac{G + \left(\frac{p_o\left(1 - \frac{c}{L_2}\right) - 4\,W}{p_o - 2\,W}\,N - F\right)\frac{D_s}{D_c}}{J + (2\,N - F)\,\frac{D_s}{D_c}}\,L_2 \quad \ldots\ldots\ldots\ldots (44)$$

Case II.—Determination of $\frac{\partial\, V_c}{\partial\, b}$.—Let L_b equal the bending span for the deck slab. Then,

$$L_b = 2\left(L_1 + \frac{a}{2}\right) = \frac{2}{p_o}\,(p_o - W)\,(L_2 - b) \quad \ldots\ldots\ldots (45)$$

The maximum bending moment for the deck is expressed by,

$$M = \tfrac{1}{2}\,\frac{W}{p_o^2}\,(p_o - W)^2\,(L_2 - b)^2 \quad \ldots\ldots\ldots\ldots\ldots (46)$$

The deck thickness, d_2, may now be determined from the usual formula,

$$M = K\,b\,d_2^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (47)$$

Combining Equations (46) and (47) and solving for d_2 will result in the equation,

$$d_2 = \frac{p_o - W}{p_o}\,\sqrt{\frac{W}{2\,K}}\,(L_2 - b) \quad \ldots\ldots\ldots\ldots\ldots (48)$$

Introducing this value of d_2 in Equation (17) will result in an equation identical with Equation (23), but in which:

$$B' = \frac{W}{p_o - 2\,W}\left[4\,K_1\,A + 2\,s + (p_o - W)\,\frac{p_o}{W^2}\,\sqrt{\frac{W}{2\,K}}\left(1 + \frac{c}{L_2}\right)\right] \quad \ldots\ldots (49)$$

and,

$$G' = -\frac{p_o - W}{p_o - 2W}\frac{p_o}{W}\sqrt{\frac{W}{2K}}\left(1 + \frac{c}{L_2}\right) + 2K_1 A \frac{p_o - 4W}{p_o - 2W} + 2s \ldots (50)$$

Differentiating V_c with respect to b as before, will result in an equation identical with Equation (27), but in which the values of B', G', and J, in Equations (49), (50), and (26), respectively, are substituted. The volume of steel in the haunch will have the same value as before under Case I.

Case II.—Determination of $\frac{\partial V_d}{\partial b}$.—The steel area in the deck required for bending moment will be,

$$R_d = \frac{1}{j_2 f_s}\frac{p_o - W}{p_o}\sqrt{\frac{WK}{2}}(L_2 - b) \ldots\ldots\ldots\ldots\ldots (51)$$

The volume of the steel in the deck will then have the value,

$$V_d = \tfrac{1}{2}(p_o - 2W)\frac{W}{p_o^2}F'(L_2 - b^2) \ldots\ldots\ldots\ldots\ldots (52)$$

in which,

$$F' = 2(1 + e_2)\frac{p_o - W}{p_o - 2W}\frac{p_o}{W}\sqrt{\frac{W}{2K}}\frac{K}{j_2 f_s} \ldots\ldots\ldots\ldots (53)$$

Differentiating:

$$\frac{\partial V_d}{\partial b} = -(p_o - 2W)\frac{W}{p_o^2}F'(L_2 - b) \ldots\ldots\ldots\ldots\ldots (54)$$

Substituting these values for $\frac{\partial V_c}{\partial b}$, $\frac{\partial V_h}{\partial b}$, and $\frac{\partial V_d}{\partial b}$, in Equation (14) and solving for b, an equation identical to Equation (44) is derived, but in which F' from Equation (53) is substituted.

Equation (44) determines the value of b that will result in the most economical design. It will be noted, that for a constant head the ratio, $\frac{b}{L_2}$, will be independent of the clear buttress spacing. Therefore, for a certain set of stresses and values of the coefficients, this ratio may be solved as a function of the height, H, and then used for any buttress spacing desired.

In the following is presented a typical example, illustrating the solution of Equation (44) for varying heights of the dam and when using values of stresses and coefficients that are customary for this type of structure.

The following stresses and values of the coefficients have been used: $p_o = 600$ lb per sq in.; $f_c = 650$ lb per sq in.; $f_s = 16\,000$ lb per sq in.; $\tau = 120$ lb per sq in.; $v = 120$ lb per sq in.; $\sigma_1 = 500$ lb per sq in.; $K = 107.7$; $j_1 = 0.875$; $j_2 = 0.875$; $\mu = 0.33$; $e_1 = 0.5$; $e_2 = 0.2$; $K_1 = 1.1$; and $\frac{D'_s}{D'_c} = 0.005$, in which, D'_s is the cost of steel, in dollars per pound, and D'_c is the cost of concrete, in dollars per cubic yard. Then $\frac{D_s}{D_c} = 13\,200\frac{D'_s}{D'_c} = 66$.

The loading on the deck, W, in pounds per square inch, may be written,

$$W = 0.434\ H \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (55)$$

in which, H is the height, in feet, from the under side of the deck to maximum high-water level. For a general case, the buttress thickness, $2c$, may be determined approximately from the equation,

$$(L_2 + c)\ W = c\sigma_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (56)$$

in which, σ_1 is the first principal stress in the buttress. Solving for $\frac{c}{L_2}$ there results,

$$\frac{c}{L_2} = \frac{W}{\sigma_1 - W} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (57)$$

The maximum allowable bending stresses will determine the deck thickness at the top of the dam. The height of dam below which shear governs the deck thickness may be determned by combining Equations (21) and (48), which will result in the equation,

$$\frac{W}{\nu j_2} \frac{(L_2 - b)\ (p_o - 2\,W)}{p_o} = \frac{p_o - W}{p_o} \sqrt{\frac{W}{2\,K}}\ (L_2 - b) \quad \ldots\ldots\ldots (58)$$

This height does not depend upon the span as the factor, $(L_2 - b)$, will disappear. Solving for W and remembering that $W = 0.434H$, the value of the height below which the shear governs the deck thickness will be for the conditions assumed, namely, $H = 155$ ft. Therefore, b may be determined from Equation (44), with substitution factors, G, J, N, and F, for $H < 155$ ft, and from Equation (44) for $H > 155$ ft, with substitution factors, G', F', J, and N. The results obtained by solving Equation (44) are shown graphically by Fig. 5, for five values of s. For a certain slope of the under side of the haunch, these curves will give directly the most economical ratio of $\frac{b}{L_2}$.

Fig. 5 shows how the ratio, $\frac{b}{L_2}$, for a constant haunch slope is increasing with the head. This indicates that the use of enlarged haunches and shortened deck will be more economical as the height of the dam increases until, finally, below a certain height the best economy will be obtained by omitting the deck entirely and using the round-head type. These limiting heights, below which the round-head type is the most economical, are indicated by horizontal dashed lines in Fig. 5.

The equation for total cost, C_t, of haunch and deck will be of the second degree with respect to b. Therefore, it will have only one minimum and C_t will increase continuously if the value of b is departing from that determined by Equation (44), or as shown by Fig. 5. Thus, if for practical reasons this value of b cannot be used, the best economy will be attained by having the actual value of b approach as nearly as possible the value of b determined from the equations.

A comparison of the cost of the water-bearing members for different values of the ratio, $\frac{b}{L_2}$, would indicate the saving effected by using values as determined by the equations. It may be done by introducing the values, V_c, V_h, and V_d, in Equation (13) from which the total cost, C_t, may be solved under the different assumptions. This equation will be of the form,

$$C_t = \left(1 + \frac{c}{L_2}\right) i_2 D_c L_2 + f\left(\frac{b}{L_2}\right) L_2^2 \ldots\ldots\ldots\ldots\ldots (59)$$

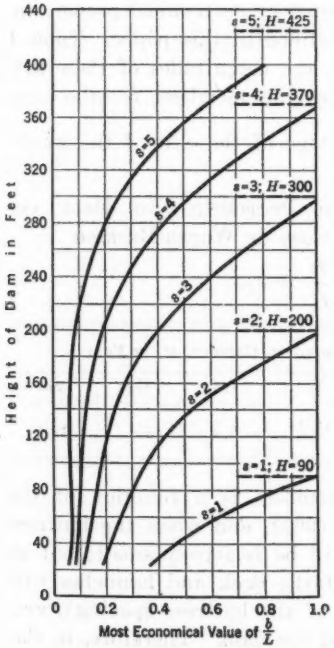

FIG. 5.—MOST ECONOMICAL VALUE OF $\frac{b}{L_2}$.

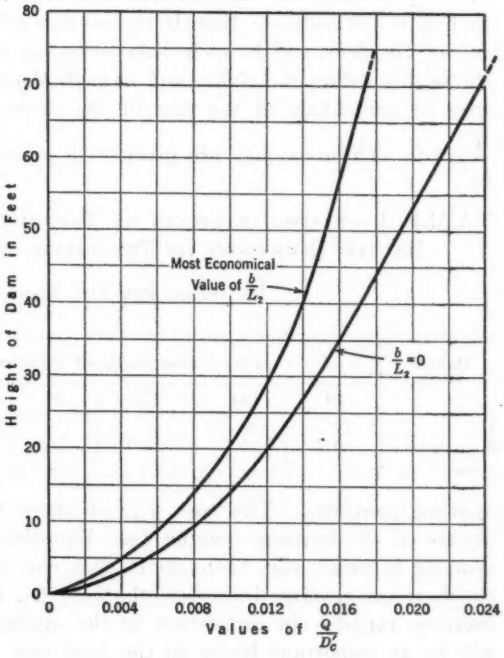

FIG. 6.—VALUES OF Q FOR $s = 1$.

The first term for the value of C_t will be small in comparison with the second one; also, it will be independent of the ratio, $\frac{b}{L_2}$, and, therefore, it may be omitted for this comparison; hence,

$$C_t = f\left(\frac{b}{L_2}\right) L_2^2 = QL_2^2 \ldots\ldots\ldots\ldots\ldots\ldots (60)$$

Fig. 6 shows the values of Q for $s = 1$, the values of Q being for the following values of the ratio, $\frac{b}{L_2}$: First, $\frac{b}{L_2} = 0$; that is, the tongue has the

same thickness as the buttress (Fig. (1(*a*)); and second, $\frac{b}{L_2}$ as determined from Equation (44). This coefficient will give the minimum cost when introduced in Equation (60). Curves similar to Fig. 6 for other values of s can be prepared as the need occurs.

When the cost of concrete, D'_c (in dollars per cubic yard), is known, the coefficient, Q, is determined by the curves in Fig. 6. Introducing this value of Q in Equation (60), this equation will give the cost, C_t, directly in dollars per foot length of deck and haunch for any span, L_2, desired (L_2 = one-half the clear buttress spacing, in feet). Therefore, the curves in Fig. 6 will indicate directly the savings effected by economical proportioning of the deck and haunch dimensions, as outlined in this paper. Table 1 shows, for different heights and haunch slopes, the magnitudes of these savings in percentage of the cost of the water-bearing members for the case, $\frac{b}{L_2} = 0$. These savings are given as a percentage of the cost of the water-

TABLE 1.—SAVINGS EFFECTED BY ECONOMICAL PROPORTIONS OF DECK AND HAUNCH DIMENSIONS (IN PERCENTAGE OF COST OF WATER-BEARING MEMBERS, FOR THE CASE, $\frac{b}{L_2} = 0$).

Haunch slope, s	PERCENTAGE SAVINGS CORRESPONDING TO HEIGHTS, H, IN FEET							
	24	48	72	96	120	144	192	240
1	17	20	27	...	...	.....	...	....
3	5.3	5.2	5.0	5.2	5.2	5.4	8.1	12.8

bearing members. The cost, C_t, of these members is a function of the square of the buttress spacing (see Equation (60)), and when the buttress spacing is small, say, 15 to 20 ft, this cost will be relatively small; but as the buttress spacing increases, the cost C_t, of the deck and haunches will increase rapidly (in proportion to the square of the buttress spacing), and will be an important factor in the total cost of the dam. Therefore, in the case of wide buttress spacing, the savings in cost of the water-bearing members made possible by proportioning them according to the procedure outlined in this paper will materially affect the total cost of the dam.

CONCLUSIONS

The equations developed in this paper are sufficient for the precise determination of the most economical proportions of the decks and haunches of articulated buttress dams. The most economical relation between the span of the deck and the clear buttress spacing is shown to be independent of the buttress spacing for a given height of dam. Furthermore, the quantities of concrete and steel in the decks and haunches, as well as the total cost of these members, are shown to be in direct proportion to the square of the buttress spacing. The examples clearly indicate that for dams of great

height for which the buttress spacing generally is large, the savings effected by the proportioning as outlined are of major economical importance.

Although the mathematical developments may seem formidable the results as presented in the final simplified equations and curves may be applied easily and quickly to actual design problems. Fortunately, the flexible, articulated buttress dam is subject to an exact mathematical analysis of both economy and stresses, as there are no statically indeterminate elements introducing mathematical and structural uncertainties.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FORESTS AND STREAM FLOW

Discussion

BY W. G. HOYT, M. AM. SOC. C. E., AND H. C. TROXELL, ASSOC. M. AM. SOC. C. E.

W. G. HOYT[65], M. AM. SOC. C. E., AND H. C. TROXELL[66], ASSOC. M. AM. SOC. C. E. (by letter).[66a]—The earnest consideration given to this paper, as indicated by the numerous contributions submitted, is appreciated. It is also extremely gratifying that, with one or two exceptions, the discussions have either agreed with the conclusions and deductions of the writers or have contained constructive criticism.

Unfortunately, Mr. Bates looks upon the paper as an attack upon the forests and the forester instead of a contribution to the science of hydrology. He discusses soil, geology, meteorology, and "Nature's function" in an attempt to prove that the facts found in specific areas are not generally applicable. Reliable statistical information, however, collected under rigid scientific control in areas presenting widely different conditions, must be held to outweigh opinions such as he presents, no matter how firmly held or enthusiastically advocated.

No question has been raised by any member of the Society as to the method of analysis used, and no one has disputed the conclusions that on the two areas under consideration: (1) Forests and brush cover substantially lower stream flow at practically all seasons and all stages, including flood, minimum, and total flow; (2) forests and brush cover substantially lessen erosion where readily erodible material is available; and, (3) new vegetative growth exercises an effect similar to the original cover in retarding erosion, but without so detrimental an effect on annual run-off and summer low-water flow.

NOTE.—The paper by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxell, Assoc. M. Am. Soc. C. E., was presented at the Annual Convention, Yellowstone National Park, Wyoming, July 6, 1932, and was published in August, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1932, by C. G. Bates, Esq.; November, 1932, by Messrs. J. E. Willoughby, and A. L. Sonderegger; December, 1932, by Messrs. J. C. Stevens, Harry F. Blaney, Daniel W. Mead, Ralph R. Randell, H. K. Barrows, Donald M. Baker, Ralph A. Smead, and George H. Cecil; February, 1933, by Messrs. C. W. Sopp, and W. B. Rowe; March, 1933, by Messrs. W. C. Lowdermilk, Rhodes E. Rule, and Robert E. Kennedy; April, 1933, by Messrs Herman Stabler, and H. S. Gilman; and May, 1933, by Charles H. Lee, M. Am. Soc. C. E.

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[66a] Received by the Secretary, July 27, 1933.

Messrs. Lowdermilk and Gilman deplore, as do the writers, the absence of precipitation records in the Southern California area. Hydraulic engineers will realize readily, however, that the method of analysis used is independent of rainfall records, but takes into account all normal differences in meteorological conditions that are reflected in run-off from the two areas.

Additional information concerning the climatic conditions and the geology would be most helpful in a search for the relative influence of causes contributing to the results. The lack of such information does not impose a serious limitation on the validity of the conclusions, nor does it cast doubt on the aforementioned facts.

However, the fact that Mr. Gilman apparently believes that the precipitation record for Mount Wilson had been used in computing the normal run-off of Fish Creek indicates that the method of determining the normal run-off, described in the paper, may not be fully understood by all readers.

Students of hydrology in Southern California have long known that there is not a simple direct relationship between run-off and rainfall. For example, a 2-in. rainfall, if concentrated within an hour, may cause flood run-off, but the same rainfall distributed over 24 hours would cause little if any run-off. For this reason, unless the intensity of the rainfall and its relation to run-off are known, the rainfall record is of little value in determining the normal run-off of Fish Creek.

The daily relationship of the discharge for Santa Anita and Fish Creeks was plotted for each day for the period prior to the fire (1917–1924). These points indicated a fairly well-defined, simple, direct relationship of the run-off from the two areas, as exemplified in the following tabulation, which records various simultaneous flows (in second-feet per square mile) that occurred during the winter months of January to March, 1917 to 1924:

Daily discharge, Santa Anita Creek	Daily discharge, Fish Creek	Daily discharge, Santa Anita Creek	Daily discharge, Fish Creek
40	82	2	2.6
20	37	1.0	1.2
10	16.5	0.5	0.58
5	7.4	0.2	0.21

The relationship was so well-defined for the period prior to the fire that there is good reason to believe that it would have been maintained for the period following the fire if the area had not been burned over. This relationship, wholly independent of records of precipitation, was used to compute the normal run-off from Fish Creek after the fire.

The rainfall records from Mount Wilson and Santa Anita Ranger Stations were included merely to show how little of the rainfall appeared as run-off at the gauging stations. For example, in Southern California, a seasonal rainfall of 20 in. produces, as a rule, a run-off of less than 2 in., and a seasonal rainfall of 30 in., generally less than 10 in.

Mr. Lowdermilk questions whether there is any relationship in rainfall for the drainage areas of Fish Creek and Santa Anita Creek and presents

data showing the variation in seasonal rainfall at points ranging from less than 1 000 ft to more than 5 000 ft in altitude, and from less than 1 mile to more than 15 miles, from these drainage areas.

In Southern California many storms vary with the altitude; some affect only the higher altitudes, some produce rain only in the lower reaches, and others are general, giving much the same rainfall at all altitudes. Furthermore, storm rainfall varies from west to east and from east to west, depending on the direction from which the storm approaches. Thus, it would scarcely be expected that even an approach to uniformity in seasonal rainfall could be attained.

On Fig. 17 are plotted the relationships between storm rainfall for the period, 1917–1930, for Mount Wilson, for the San Gabriel power-house, near the mouth of Fish Creek, and for the Sierra Madre, just west of the mouth of Santa Anita Creek. For the lesser storms, Mount Wilson has sometimes more rainfall than the other stations and sometimes less, but as the storm rainfall increases, the relationship is better defined. In almost every storm the rainfall at the Sierra Madre, although generally greater than at the power-house, bore a relationship to that at Mount Wilson substantially parallel to that borne by the rainfall at the San Gabriel power-house. These comparisons should resolve Mr. Lowdermilk's doubt as to the general similarity of rainfall graphs for the two areas; but, after all, the most convincing evidence of rainfall relationship between these areas for which a direct record is meager is found in the well-defined run-off relationship, for which the direct record is ample, continuous, and areally integrated.

The additional detailed information by Mr. Lowdermilk showing the composition of chaparral cover in Southern California is a welcome contribution to the discussion. It is regretted that the writers did not have access to unpublished Forest Service records, and that, therefore, their description of vegetation was incomplete, but after all the names of vegetative species and knowledge of their characteristics, like records of rainfall, are useful aids to interpretation of results but cannot change records of run-off.

Readers should realize that the Wagonwheel Gap experiments were extensive and exhaustive and that they cost tens of thousands of dollars. The Southern California study cost almost nothing, being incidental to routine stream gauging. The latter study lacked many of the details of rainfall, temperature, soil cover, and sediment observations of the former and, therefore, failed to afford as extensive a field for study of the relative effects of underlying causes. The effects of change in cover on run-off, however, were recorded as effectually in the Southern California area as in Colorado, and the major conclusions may be drawn with as much assurance on one area as on the other.

Facts or conclusions as to small specific areas have little value unless their application to other areas can be fairly well demonstrated. Mr. Bates presents the view that the Wagonwheel Gap and Southern California areas are so different from the greater proportion of forested areas in the United States that no conclusion generally applicable can be drawn from them.

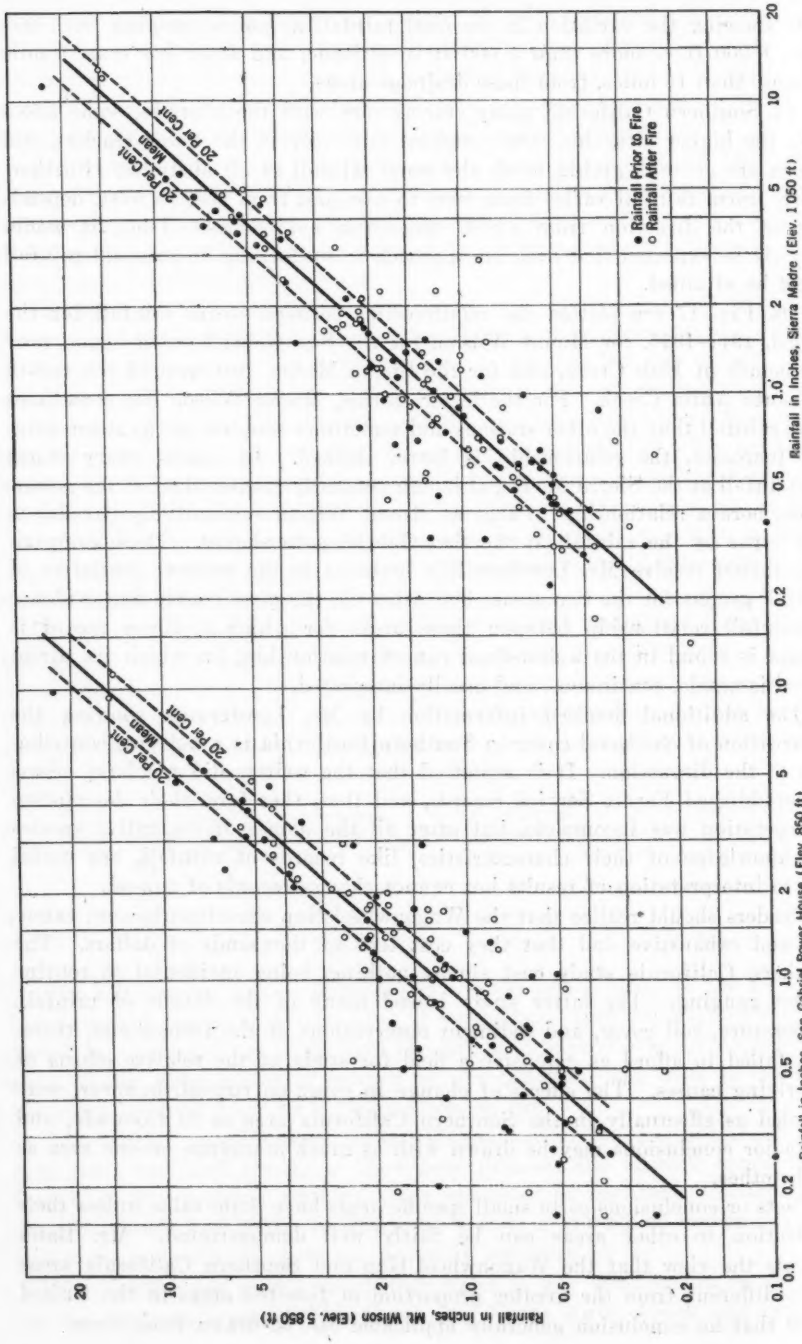
Rainfall in Inches, Mt. Wilson (Elev. 5 850 ft)
Rainfall in Inches, San Gabriel Power House (Elev. 850 ft)
Rainfall in Inches, Sierra Madre (Elev. 1 050 ft)
20 Per Cent
Mean
20 Per Cent
20 Per Cent
Mean
20 Per Cent
● Rainfall Prior to Fire
○ Rainfall After Fire

FIG. 17.—RELATION BETWEEN STORM RAINFALL, AT MT. WILSON, SAN GABRIEL POWER HOUSE, AND THE SIERRA MADRE.

In so far as the Wagonwheel Gap area is concerned the writers must dissent from this view. In 1922, Mr. Bates stated[67]:

"It was, therefore, with a full realization of the need for experimental proof and in the hope of satisfying the most critical that in 1909 the Forest Service undertook to initiate a project which should produce results bearing particularly on the conditions typical of vast areas then included in national forests of the West."

In seeking areas where studies should be made, the following factors were considered by the Forest Service: Areas should be (1) contiguous; (2) small; (3) representative of a fair average for the National forests of the Rocky Mountain region; and (4) accessible. Credit for the discovery of such an area in Wagonwheel Gap is given to a Forest Assistant, and after the area had been visited by several Forest officials, including the Assistant Forester, its selection was approved. After close acquaintance with the area for more than twelve years Mr. Bates considered that "excellent judgment" had been used in its selection for the purpose of satisfying the most critical of the correctness of a preconceived conclusion. It scarcely seems fair to the public, when that conclusion is disproved in part, to suggest now that the area is extreme and not fairly representative.

Many of the other discussers take what seems to be a more rational attitude—that identical results in Colorado and California under such widely different circumstances constitute evidence of the probable universality of the conclusions, in the absence of trustworthy experiments to the contrary. All have stressed the desirability of additional exhaustive studies, and in that opinion the writers concur fully. Experiments in a humid area and on a fairly impervious soil are needed especially.

By processes of deduction, Messrs. Lowdermilk, Lee, and others, conclude that the increase in the summer flow from the denuded Colorado area resulted from decreased interception by the forest cover, and that the corresponding increase in the Southern California area resulted from decreased transpiration. No attempt was made in the paper to explain in detail the reasons for the increase, and, in the light of present knowledge, the writers do not believe that explanation is altogether possible or, if possible, that it greatly matters. All vegetation intercepts; all vegetation transpires. Removal of vegetation will diminish both interception and transpiration. If it is true that interception was in the ascendency in the Colorado area and that transpiration was in the ascendency in the Southern California area, that is just another of those differences in regional conditions that suggest the universality of conclusions that were identical for the two fields of experiment. The essential and undisputed fact still remains that with a diminution of the vegetative cover the annual flow and the low-water flow increased on both areas.

Additional studies have been made, however, and from these the writers believe that the deductions of Messrs. Lowdermilk and Lee are not entirely correct. To the best of their ability the writers have separated the run-off

[67] "Stream Flow Experiment at Wagonwheel Gap, Colorado; Preliminary Report on Termination of First Stage of Experiments," *Monthly Weather Review*, Supplement 17, p. 2.

FIG. 17.—RELATION BETWEEN STORM RAINFALL; MT. WILSON, SAN GABRIEL POWER HOUSE, AND THE

caused by summer storms from the ordinary non-storm run-off for three years prior to deforestation and for three years afterward. Some of the results of these studies are shown graphically in Figs. 18 to 21, inclusive; others are shown in Table 22.

TABLE 22.—STORM RUN-OFF AND RAINFALL, IN INCHES, JULY AND AUGUST, AT WAGONWHEEL GAP, COLORADO

Season	Area *A*			Area *B*			Ratio, storm run-off, Area *B* to Area *A*
	Rainfall	Storm run-off	Ratio	Rainfall	Storm run-off	Ratio	
(*a*) BEFORE DEFORESTATION OF AREA *B*							
1912	5.86	0.06441	0.010	5.81	0.05088	0.0088	0.790
1913	4.67	0.04415	0.0095	4.94	0.03902	0.0079	0.884
1914	7.10	0.09504	0.0134	7.47	0.05877	0.0079	0.618
(*b*) AFTER DEFORESTATION OF AREA *B*							
1920	3.32	0.03109	0.0094	2.74	0.01927	0.0070	0.620
1921	6.42	0.04653	0.0072	6.25	0.02758	0.0044	0.593
1922	5.40	0.07170	0.0133	4.88	0.06044	0.0124	0.844

The data in Table 22 fail to disclose that increase in summer storm run-off resulting from rainfall which should follow decrease in interception by reason of deforestation. The hydrographs in Figs. 18 to 21 are especially instructive in showing the increased non-storm summer flow after deforestation, a result that should logically follow decreased transpiration. It is clearly evident that there was more ground-water available to sustain summer run-off in the Wagonwheel Gap area after deforestation than before, and it is more than likely that both decreased interception and decreased transpiration contributed to this result, although the latter seems to have been the major cause.

Mr. Lowdermilk may also have gone too far in his assumption that all the gain in summer flow after the burning of the Southern California area resulted from decreased transpiration by canyon-bottom vegetation. Mr. Blaney's work in Coldwater Canyon showed that canyon-bottom vegetation during the 6-month period used 45 acre-in. per acre, or 238 acre-in. per mile of canyon. On the assumption that transpiration by riparian vegetation above the falls in Fish Creek Canyon proceeded before the fire at the rate per linear mile indicated for Coldwater Canyon, that such transpiration was wholly eliminated after the fire, and that there were no compensating water losses by reason of increased evaporation, it might be freely admitted that change in riparian vegetation could account for the entire recorded change in summer run-off. The first of these assumptions is speculative, however, and the others are untenable. Riparian vegetation would recover rather quickly from the effects of the fire, and if its destruction had caused increased summer flow this effect would soon be lost.

The increase in summer flow, however, was well sustained for several years, suggesting that no such close relationship existed between cause and effect here as was assumed by Mr. Lowdermilk. Moreover, there would be undoubted although quantitatively unknown increased evaporation to com-

pensate in part for the effects of decreased transpiration by riparian vegetation. The continued maintenance of summer flow strongly suggests that these factors—transpiration and evaporation—were in fact compensating to a considerable degree. In the light of all the available facts, it seems that decreased transpiration was only one of several factors contributing to increased summer flow after the fire.

Because of the many uncertainties involved the writers have refrained from attempting to distribute the source of the summer gain between the burned-over canyon-bottom plant life and the chaparral-covered canyon walls in Southern California, or between the interception losses, transpiration losses, and increased ground storage in the Wagonwheel Gap area.

Mr. Lee presents an excellent discussion of the part played by forest litter on run-off and absorption. Forest litter also forms the basis for the increased absorption in Mr. Sonderegger's "super-forestation." The recent experiments by Mr. Lowdermilk[68] have contributed to knowledge of the effect of litter on stream flow. In his discussion, Mr. Lowdermilk cites these experiments as a basis for the following statement:

"The forest cover, especially the litter which it produces, furnishes conditions which make for maximum possible water and erosion control on watershed surfaces."

In so far as erosion is concerned the writers agree with this statement. They question, however, the use of the word, "water," in this broad conclusion if by "control" Mr. Lowdermilk implies that the presence of litter is reflected in an increased water yield, or increased low-water flow.

His studies show clearly that litter decreased direct run-off and, therefore, must have increased absorption. As Mr. Lee points out, however, part of the water passing into the ground is only "temporarily absorbed," being held in the zone of aeration, from which it escapes into the atmosphere by evaporation or transpiration from vegetation. The remainder is "permanently absorbed" penetrating to the water-table and entering the zone of saturation. This part is the ground-water storage that feeds stream flow. Mr. Lowdermilk fails to record the quantity of absorption that passed through the soil column in his tanks, and even if it had been so recorded it would not be indicative of Mr. Lee's "permanent absorption" under natural conditions, because the tanks were entirely artificial, in that they contained no vegetative cover to transpire what under natural conditions would have been "temporarily absorbed."

To have a litter-covered soil in Nature requires a canopy of brush or trees. As stated by many contributors, interception by and evaporation from such a canopy may range from 10 to 40% of the rainfall. Under natural conditions this interception might readily offset any increased absorption such as is inferred from results obtained in barren, artificially litter-covered tanks. Furthermore, the tanks used by Mr. Lowdermilk were set at a 30% slope, whereas in the Southern California area, as shown in Table 23, the slopes are much greater.

[68] "Influence of Forest Litter on Run-Off, Percolation, and Erosion," by W. C. Lowdermilk, *Journal of Forestry*, Vol. 27, No. 4, April, 1930.

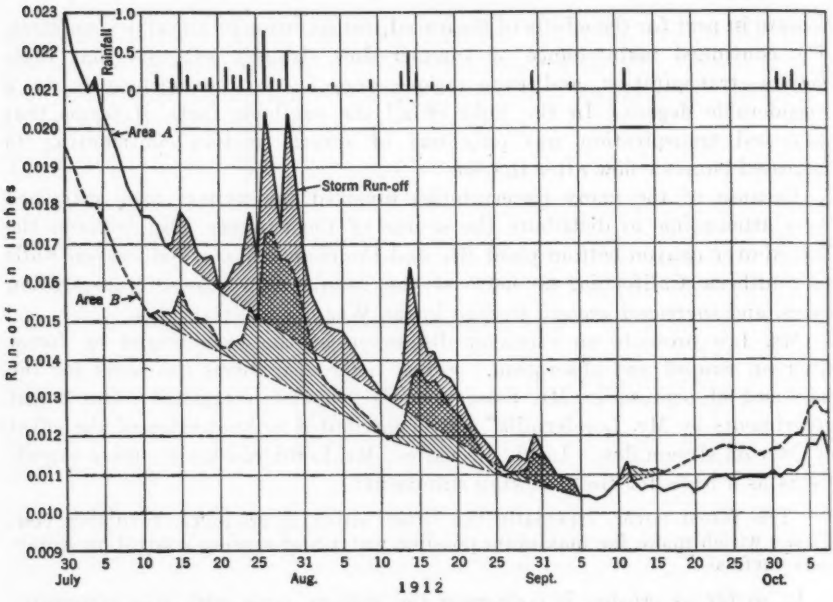

FIG. 18.—STORM RUN-OFF, WAGONWHEEL GAP, COLORADO, JULY TO OCTOBER, 1912 (BEFORE DEFORESTATION).

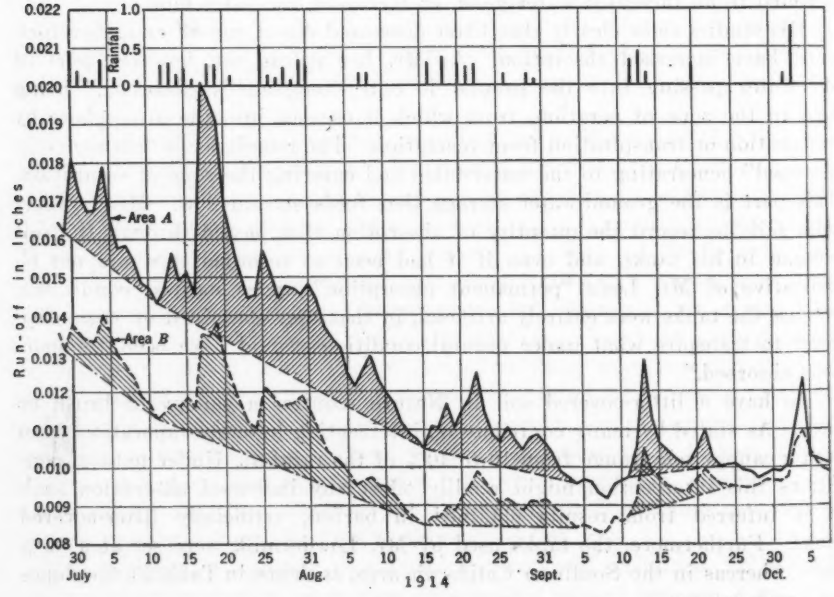

FIG. 19.—STORM RUN-OFF, WAGONWHEEL GAP, COLORADO, JULY TO OCTOBER, 1914 (BEFORE DEFORESTATION).

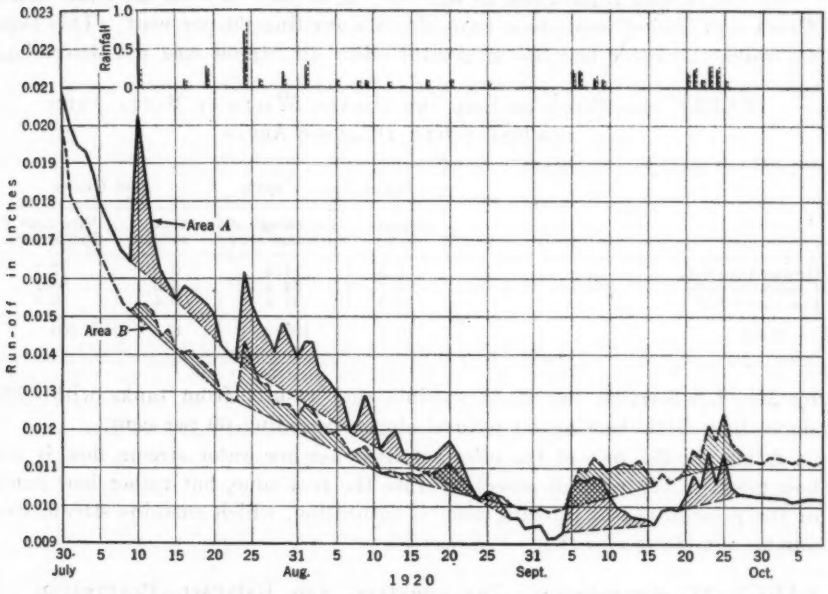

FIG. 20.—STORM RUN-OFF, WAGONWHEEL GAP, COLORADO, JULY TO OCTOBER, 1920 (AFTER DEFORESTATION OF AREA *B*).

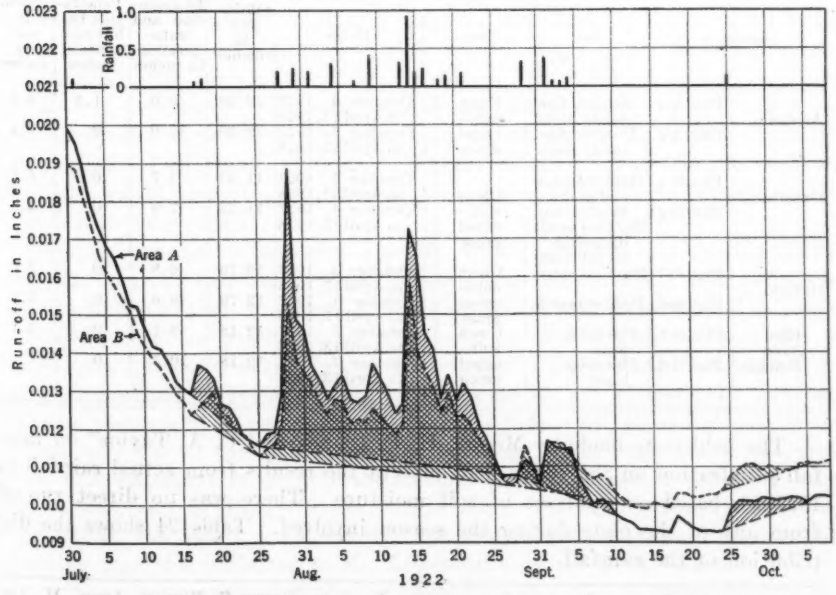

FIG. 21.—STORM RUN-OFF, WAGONWHEEL GAP, COLORADO, JULY TO OCTOBER, 1922 (AFTER DEFORESTATION OF AREA *B*).

It will be seen from Table 23 that the major parts of both the Santa Anita Creek and Fish Creek areas have slopes exceeding 60 per cent. This slope at which the litter has the maximum effect on run-off was not determined

TABLE 23.—SLOPE OF SOIL AND CANYON WALLS OF SANTA ANITA AND FISH CREEK DRAINAGE AREAS

Slope	Santa Anita Creek		Fish Creek	
	Square miles	Percentage of drainage area	Square miles	Percentage of drainage area
Greater than 80%	1.5	14.3	0.5	7.7
From 80 to 60%	5.7	64.3	5.6	86.2
Less than 60%	3.3	31.4	0.4	6.1
Total	10.5	100	6.5	100

by Mr. Lowdermilk, but it is possible that results from tanks with 30% slopes have little bearing on natural slopes exceeding 60 per cent.

After all, the test of the effect of litter on low-water stream flow is not how much of the rainfall percolates into the root zone, but rather how much of the rainfall passes into the zone of saturation, which sustains stream flow during non-storm periods.

TABLE 24.—EVAPORATION, TRANSPIRATION, AND RAINFALL PENETRATION IN SOUTHERN CALIFORNIA

Location		Soil	Crop	Dates	Rainfall, in inches	Evaporation and transpiration, in inches	Penetration below root zone, in inches	Stored in soil, in inches
Anaheim..	Plot A-1.	Hanford fine sandy loam	Clean cultivated	October 1, 1927 to April 15, 1928	12.39	6.0	1.5	4.9
	Plot A-2.	Hanford fine sandy loam	grass	October 1, 1927 to April 15, 1928	12.39	12.0	0	0.4
Ontario...	Plot D-1.	Hanford sand underlaid by silt loam	Clean cultivated grass	October 1, 1927 to April 17, 1928	11.52	4.7	0	6.8
	Plot D-2.	Hanford sand underlaid by silt loam		October 1, 1927 to April 17, 1928	11.52	7.9	0	3.6
Ontario...	Plot E-1.	Hanford sand	Clean cultivated grass	October 1, 1927 to April 17, 1928	12.79	6.8	0	6.0
	Plot E-2.	Hanford sand		October 1, 1927 to April 17, 1928	12.79	9.6	0	3.2
Glen Avon Heights	Plot G-1.	Placentia loam	Clean cultivated vetch	October 1, 1927 to April 13, 1928	12.18	8.1	0	4.1
	Plot G-2.	Placentia loam		October 1, 1927 to April 13, 1928	12.18	10.5	0	1.7

The field tests made by Messrs. H. F. Blaney and C. A. Taylor[69] on rainfall penetration on the valley floor present the results from actual rainfall in 1927–28, based on analyses of soil moisture. There was no direct run-off from any of the plots during the season involved. Table 24 shows the distribution of the rainfall.

[69] "Rainfall Penetration and Consumptive Use," by Harry F. Blaney, Assoc. M. Am. Soc. C. E., and C. A. Taylor, *Water Resources Bulletin 33*, California Dept. of Public Works, 1930.

In each of the tests noted in Table 24, more rainfall was stored in the clean cultivated plots than in those bearing vegetation. The depth of the root zone was from 6 to 11 ft, and the initial deficiency of soil moisture in the root zone ranged from 4.6 in. on Plot A-2 to 13.2 in. on Plots D-1 and D-2. As soon as the initial deficiency of soil moisture is overcome, rainfall penetrates below the root zone and becomes a contribution to ground-water.

The soil conditions, as so ably outlined by Messrs. Mead and Lee, would have a major effect on storm run-off and absorption. On soils of the more pervious material, forests have little effect on direct run-off. This is amply illustrated by Figs. 18 to 21, which show that the removal of the forest cover in the Wagonwheel Gap experiment did not perceptibly increase the storm run-off following summer rains.

"Super-forestation," as Mr. Sonderegger states, would "keep the water in the water-shed," but retarding storm run-off by means of plant life does not imply conservation of water. As Mr. Rule states, it results in "a reduction in the residual quantity available for capture by Man."

Under the climatic conditions existing in the Santa Ana River Valley and the Coastal Plain in California, according to Messrs. Blaney and Taylor:[70]

"The results of the last three years' work on brush plots indicate that a seasonal precipitation of at least 19 inches is necessary before any material amount of water will penetrate below the brush root zone of the valley floors. A seasonal rainfall of less than 19 inches is usually consumed by the brush-covered area before the ground water receives any increment."

Messrs. Blaney and Taylor also show that for equal rainfall more of the rainfall passes below the root zone in bare soil than in soil covered with plant life and that deficiency of soil moisture is much less in bare soils than in brush-covered areas.

Recent excavation for roads in chaparral areas in Southern California has uncovered root systems that have extended as deep as 30 ft into shattered rock in search of water. Only during years of exceptional rainfall, such as 1921–22, could important accretions from rain be made to ground-water supplies in this region.

The writers are in harmony with Mr. Rule's valuable contribution showing the relation existing between vegetal cover and storage soil moisture. The facts brought out in several of the discussions tend to confirm Mr. Rule's suggestion that in so far as the Southern California area is concerned a large part, but not necessarily the major part, of the increased summer flow may be credited to reduction in the consumptive use of water by canyon-bottom vegetation. The writers stated, that "some of the alders and sycamores adjacent to the stream escaped burning." Mr. Lowdermilk declares that this statement is entirely misleading, as this condition prevailed only below the falls in Fish Canyon, while above the falls all canyon-bottom vegetation was killed. Apparently, less canyon-bottom vegetation escaped burning than was indicated to Mr. Lowdermilk by the word, "some."

[70] "Rainfall Penetration and Consumptive Use of Water in Santa Ana River Valley and Coastal Plain," by H. F. Blaney, Assoc. M. Am. Soc. C. E., and C. A. Taylor, *Bulletin 33*, California Dept. of Public Works, 1930, p. 21.

The studies by Messrs. Blaney, Taylor, Nickle, and Young, as described by Mr. Blaney, are of outstanding value in showing the extent of water losses through transpiration by hydrophytic, or more properly phreatophytic, canyon-bottom vegetation. These studies show gains of more than 100% in summer flow through the elimination of transpiration losses in canyon bottoms having widths of only 30 to 50 ft. It seems apparent that considerable gains could be obtained through the reduction of similar losses now occurring in streams, such as the Santa Ana and others which meander for miles through wide brush and tree-covered areas. Pumping ground-water into conduits constructed across such areas, thus lowering the water-table below the root zone and reducing losses by evaporation and transpiration, may be economically sound in areas where water-rights are worth between $100 000 and $200 000 per sec-ft.

Mr. Barrows touches lightly on one phase of the question which may be particularly applicable to New England and the North Atlantic region. Statements of foresters indicate that many water-sheds in this region have been heavily reforested with the sole idea of increasing water supply for municipal use. As Mr. Barrows states, and as the experiments clearly indicate, "it may be worth while for other reasons, * * *, but certainly as regards increasing the water supply the results are likely to be detrimental rather than helpful."

Mr. Baker believes that the "superficial run-off + ground storage" would be a "minus" after deforestation, and, consequently, that "precipitation" in his basic formula would be reduced. Here, again, the assumption is made that increased direct run-off necessarily connotates reduction of ground-water storage. It is an important fact that on the Wagonwheel Gap area the normal flood run-off was held temporarily as ground-water. The flood run-off increased after deforestation; consequently, more water must have entered the soil column after deforestation. The work of Messrs. Blaney, Taylor, and their associates in Southern California, already referred to, seems to substantiate this point.

Mr. Baker apparently accepts the conclusion as sound for areas having a "marine type" of precipitation, but raises a question as to the possible detrimental effect of deforestation of several thousand square miles having a "continental type" of precipitation.

The effect of plant life on precipitation is still a debatable question, to which there seems to be no definite answer. It appears unlikely, however, that the loss, if any, in the "continental type" of rainfall due to a change in vegetative cover from trees to smaller species would offset the gain in water supply. It would seem as logical to suppose that the present practice of irrigation on 30 000 sq miles of arid land in the West by changing the cover from the desert type to farm crops would have an effect upon precipitation as to suppose that a change in cover on several thousand square miles in the Southeast would have a serious effect upon agriculture. Incidentally, the Wagonwheel Gap area is probably as far removed from a "marine type" of climate as any other area in the United States that could be selected.

Comment seems unnecessary with respect to claims that deforestation has changed climate or that forestation increases precipitation. These and similar claims are believed to be incapable of either proof or disproof. Some assert that removal of the cedars of Lebanon brought destructive floods and resultant erosion[71], but according to the Bible long before those cedars were removed the greatest flood of the world took place and was the direct result of abnormal precipitation, as are the floods of to-day.

The studies by Messrs. Burr, Chittenden, Harts, Hazen, Mead, Stevens, and others already cited indicate, in a general way at least, that over broad areas no decreases in stream flow can be attributed to deforestation. To their conclusions may be added those recently reached in Ohio by C. E. Sherman, M. Am. Soc. C. E., who after a study[72] of run-off measurements made by Mr. D. S. Bates for the Ohio Canal Commission in 1825, when "not one-half of the 40 000 square miles of our Ohio territory is yet settled; more than five-sixths of it remains uncleared and uncultivated," concludes, in part, as follows:

"The Ohio Canal Commission reports which give early stream records make surprising reading for those interested in the effect of forests on drought, for it is difficult to read them carefully without concluding that at many places in Ohio reforestation would have little effect in maintaining good stages of water during severe drought."

Mr. Lowdermilk states that the late Professor Swain, "deserves to rank in authority much higher than Colonel Chittenden." The writings and statements of both these distinguished and able engineers were made prior to the Wagonwheel Gap experiments. Professor Swain fully realized the need of tangible evidence when he discussed the late Colonel Chittenden's masterpiece in 1909. He states in his discussion[73] that,

"In order to demonstrate quantitatively beyond peradventure the effect of forests, it would be necessary to have the same area under identical conditions in every respect, except that at one time it should be wooded and at another time deforested."

It is possible only to surmise to what extent the opinions of both these men would have been changed, if at all, had the results of the Wagonwheel Gap and Southern California experiments been available to them. Would they have ignored the records, or would they have changed their opinions to agree with the records? To the extent that the conclusions and deductions of both agree with facts as now known, they should be accepted; to the extent that they are shown to be fallacious, they should be ignored.

Mr. Smead mentions the investigations of 1911 and 1912 in the White Mountains which were carried on by the U. S. Geological Survey under the provisions of the Weeks Act. In these investigations ten nearly contiguous areas in New Hampshire were selected. No final report of the investigations

[71] "Partnership of Wood and Water," by Hans Burger, *American Forests*, April, 1932, p. 147.

[72] "Ohio Stream Flow," Ohio State Univ. Studies, Vol. 1, No. 5, 1932, p. 164.

[73] *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 365.

has been prepared by the Geological Survey, but in a preliminary statement[74], the following conclusion was reached, based on comparisons made on two successive floods occurring on two of the ten streams during one spring period:

"The results of the Burnt Brook-Shoal Pond Brook studies are held to show that throughout the White Mountains the removal of the forest growth must be expected to decrease the natural steadiness of dependent streams, during the spring months at least."

On this conclusion rested largely the expenditure of several millions of dollars in the acquisition of National Forests in the East under the Weeks Act. Since the preliminary statement was issued considerable time and effort have been expended in a study of the original run-off records collected on the ten areas to determine whether it was possible to separate the effects of the inherent basin characteristics—soil, geology, shape, slope, and aspect—on the run-off from the effect of the vegetative cover.

To date this separation has not been possible, although apparently the inherent basin characteristics have more effect upon stream flow than the vegetative cover. In other words, observed differences may be the result of any one condition or combination of conditions, into which forest cover may not enter. This fact, which is also well supported by exhaustive studies referred to by Professor Mead and others and which is commonly accepted by all students of hydrology, renders studies without "control" substantially valueless. It is for this reason that the writers stress the value and importance of the type of studies carried on in Colorado and Southern California. In these areas changes from the normal are shown to be directly related to changes in the vegetative cover.

In all prior experiments, such as those mentioned by Mr. Smead, many described by Mr. Zon,[75] or the much quoted Swiss experiments,[76] it is impossible properly to ascribe reasons for noted differences in stream flow, because of a lack of "control." There is no doubt that further studies through the co-operative effort of the ecologist, forester, soil technologist, hydraulic engineer, and geologist, are necessary to determine the best type of vegetative cover for different types of soil, topography, and rainfall.

Such studies may well follow the method used at Wagonwheel Gap. In fact, in that area there is an excellent opportunity for an interested agency to study the effect of the new growth of aspens on stream flow as compared with the original cover, which consisted largely of conifers. The Utah experiment by Forsling referred to by Mr. Stabler was made along the lines adopted for the Wagonwheel Gap area. It was not wholly satisfactory, however, on account of the inability to hold an area in a semi-arid country uniform as to minor surface irregularities and vegetative cover. Still, as Mr. Stabler points out, the Forsling experiment tends to corroborate conclusions in the main

[74] Mimeograph report of thirty-one pages dated June 4, 1912, signed by George Otis Smith, Director.

[75] "Forests and Water in the Light of Scientific Investigation," by Raphael Zon. Forest Service, U. S. Dept. of Agriculture, 1927.

[76] "Untersuchungen über den Einfluss des Waldes auf den Stand der Gewässer," by Arnold Engler, Schweizerischen Zentralstadt für das forstliche Versuchswesen Mitteilungen, Vol. 12, 1919, Zürich, Switzerland; also "Partnership of Wood and Water," by Hans Burger, *American Forests*, April, 1932, p. 147.

paper and to extend their application. The type of experiment outlined by Mr. Blaney is satisfactory so long as it is conducted in rock-walled canyons having no ground-water inflow between the measuring sections. In areas where ground-water inflow takes place complications are introduced, which make it extremely difficult to determine losses resulting from vegetation. Controlled experiments on small plots, to which reference is made by Messrs. Bates and Lowdermilk, are unquestionably of value, but there is grave doubt as to their application to natural water-sheds in so far as water yield or net erosion is concerned.

With particular reference to conditions in Southern California, Mr. Cecil states,

"This vast sum [well over $1 000 000] is expended annually not for the purpose of protecting a commercial timber supply or for the most part even a growth of tree species. This cover is, in the main, chaparral * * * maintained on the assumption that: (1) It is a conserver of water; (2) it controls erosion; (3) it retards flood run-off; and (4) recreational value would be impaired by its removal."

That the chaparral cover "conserves water" has not been demonstrated. The question at issue, therefore, is whether chaparral or some other cover best fulfills the other intended purposes.

The balance between soil, topography, run-off, rainfall, and vegetation in natural undisturbed water-shed units does not necessarily represent optimum conditions of water supply. It is fairly well demonstrated that a minimum of vegetative cover will be reflected in a maximum water yield. Mr. Zon[16] states this same principle: "The more highly developed the vegetal cover the faster the moisture is extracted from the soil and given off into the air. In this respect, forest is the greatest desiccator of the soil." If chaparral represents a minimum vegetal cover that will diminish or retard erosion and increase the absorptive capacity of the soil, and if the cost of protecting it from fire is commensurate with the benefits received, then the community expenditure of more than $1 000 000 per year for chaparral protection is justified.

Mr. Tracy I. Storer,[17] presents evidence indicating that when burning of certain chaparral areas was practiced in California, herbaceous vegetation was plentiful, "but since burning has been prohibited herbaceous vegetation and grasses are screened out by the growth of chaparral." Apparently, some thought should be given to the advisability of continuing a policy of retaining, through rigid control at vast annual expense, an inflammable cover that is useless for man or beast when it appears that, in parts of California, at least, restricted burning of chaparral areas will permit the growth of herbaceous grasses that would have economic value for grazing and also a water-shed "protective value." The same thought is also expressed by Mr. Willoughby.

On account of the necessity for economy in cost of publication, the original paper was greatly condensed. Necessarily, in so doing, as Mr. Cecil states,

[17] "Factors Influencing Wild Life in California, Past and Present," *Ecology*, October, 1932, p. 323.

"several factors * * * that are important * * * have been overlooked entirely, and others have not been given proper value in arriving at a conclusion."

With the possible exception of erosion it is believed that in the main any omissions from the original paper have been covered in the various discussions. Although the original paper dealt principally with the effects of forests on stream flow, it was realized that the subject of erosion is intimately connected with the problem under consideration. The writers stated, that: "In the Southern California area complete denudation increased erosion as a direct result of the increased surface run-off" and "deposition of eroded material and ash carried by the streams during the first year after the fire was materially injurious to agricultural lands and transportation rights of way below the canyon." Unquestionably, the eroded material was a source of expense and inconvenience. Floods and attendant erosion, however, have been occurring in this region for centuries prior to the settlement of Los Angeles County.

E. C. Eaton, M. Am. Soc. C. E., to whom Mr. Cecil refers, states,[78]

"Take Los Angeles County as an example; it consists of a broad sloping valley area with a background of high mountains roughly equal to the valley area, from which ages of erosion have built deltas to known depths of more than 1 000 ft. * * * The tremendous area and great depth of the gravel cones are in reality of the greatest benefit to this southern County. * * * These underground reservoirs are responsible for the entire early development of the County and supply two-thirds of the water used in the County to-day."

Messrs. Cecil, Gilman, and Lowdermilk infer that the increased run-off made available through diminution of forest cover is non-usable and wholly destructive. Their inference may not be correct.

It is a well-known fact that a large part of the water supply in Southern California is pumped from underground basins and that as a means of increasing the ground-water storage spreading grounds have been established at various points, generally along the mountain fronts. Experience has shown that water which carries suspended matter is not readily absorbed on these spreading grounds. If the present spreading grounds were the sole entry of water into the natural underground reservoirs, there would be a sounder basis for the objections raised against the contentions in the paper. In discussing the question, however, several important facts have been apparently overlooked. In the first place, spreading operations are ordinarily conducted at a time when the entire stream flow would naturally be absorbed by stream channels and none wasted in the ocean. The Santa Ana spreading ground is practically the only exception, and here spreading operations are carried on only when there would be wastage of water to the ocean. It is common practice to let muddy water pass diversion gates in order to safeguard favorable conditions of spreading grounds to receive clear water.

This same muddy water, however, which seems to be waste from the viewpoint of the observer at the spreading grounds, forms one of the largest

[78] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 198.

contributions to the ground-water from the rainfall in the mountain area. Tables 25 and 26 show that muddy water can be and is passed into ground-water.

TABLE 25.—CHANNEL ABSORPTION IN SANTA ANA RIVER*

Period	Run-off near Prado, Calif., in acre-feet†	Waste into ocean, in acre-feet	Absorption, in acre-feet
April, 1926	44 700	21 300	23 400
February, 1927	94 500	57 200	37 300
March, 1927	25 100	6 150	18 950
February, 1931	25 800	7 760	18 040

* *Water Supply Papers 631, 651*, and unpublished records, U. S. Geological Survey.
† Including run-off of Santiago Creek, near Villa Park, Calif.

According to Harold Conkling,[79] M. Am. Soc. C. E., during the month of April, 1926, while 15 500 acre-ft was absorbed in the San Gabriel River channel above El Monte, only 604 acre-ft was spread at the mouth of the canyon. On the Santa Ana River more than 4 500 acre-ft of water too

TABLE 26.—CHANNEL ABSORPTION IN SAN GABRIEL RIVER*

Period	Run-off at canyon section, in acre-feet	Flow past El Monte stations, in acre-feet	Absorption, in acre-feet
March 29 to April 14, 1925	4 930	3 090	1 840
April 8 to 26, 1926	33 900	18 400	15 500
February 14 to March 11, 1927	70 400	45 400	25 000

* "San Gabriel Investigation", *Bulletin 7*, Div. of Water Rights, California Dept. of Public Works, 1929.

muddy to spread was absorbed on February 9 and 10, 1932, in a stretch of river channel not longer than 6 miles between Prado and Santa Ana, and constituted a source of ground-water supply for the Santa Ana Section. This is twice as much water as the Santa Ana Water Conservation Association spread during the entire season.

Apparently, the natural river channels below the spreading grounds play an important part in water conservation, and floods exercise an important function in maintaining these channels in condition to absorb water. Mr. Conkling states that,[80]

"The rate of percolation, particularly where the stream bed contains gravel and boulders, is much less before a major flood than after such a flood. This is due to what is generally called the disruptive effect of a flood. As the water is receding, in some years at least, a thin coat of weakly cemented material is deposited in the stream channel. * * * When a flood occurs big enough to disturb the stream bed this crust is washed away and percolation rates become a maximum."

The writers do not recommend as Mr. Bates implies, the reversal of "great constructive forces of Nature." They do, however, suggest that a change should be made in the concepts to be implied by the term "water-shed pro-

[79] "San Gabriel Investigation," by Harold Conkling, *Bulletin 5*, Div. of Water Rights, California Dept. of Public Works, 1927.
[80] *Loc cit., Bulletin 7*, 1929, p. 56.

tection." Heretofore, undue and in the main erroneous emphasis has been laid on "increasing the water supply," "securing favorable conditions of flow," and "promoting and protecting navigation." These concepts have been widely controverted, and it is evident that no general agreement can be reached in regard to them. The writers would emphasize their belief that water-shed protection as a means of "retarding erosion," "conserving soil," and "lessening torrential run-off," may be obtained by a cover of shrubs or grass, which may use relatively little water, as well as by trees, which may consume large quantities of water. This belief seems logical and scientifically sound and offers a platform on which engineers, foresters, and conservationists might fully agree. So considered, water-shed protection may be reasonably undertaken where physically and economically practicable and where it is clear that commensurate benefits will be obtained.

The Editors of the *Journal of Forestry* stated,[81]

"The Hoyt-Troxell report has had wide circulation; from several parts of the world we learn that it has misled engineers as to the value of water-shed cover for the region in which the study was conducted."

The paper has apparently aroused interest, but it is sincerely hoped that it has not misled any one as to the value of water-shed protection. The objects of the paper will have been partly attained if those concerned with problems of water-shed protection realize that the proper solution is not solely one of silvics in relation to the head-water area, but involves a knowledge of geology, hydrology, hydraulics, and water utilization, and their relation to the entire area.

The discussion of the paper has been sufficiently complete to warrant certain definite suggestions for effective methods of increasing the water yield in the Southern California area and possibly other areas without resulting detriment. Such discussion, however, seems to be beyond the scope of the paper. Because of the limits in scope the writers have also refrained from discussing many related questions which were not covered in the original paper, but which were raised by Messrs. Sopp, Cecil, Gilman, and others. The prolonged discussion of them would cloud the principal purpose of the paper, which was to present to the Engineering Profession and to scientists in general, cumulative evidence of reliable investigations under natural conditions as to effects of trees and brush. As Mr. Randell states this evidence is to the effect that "forests and brush do not increase, but instead considerably lessen, both total water supply and low-season stream flow."

[81] *Journal of Forestry*, Vol. 31, No. 3, p. 296, March, 1933.

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DISCUSSIONS

TESTS FOR HYDRAULIC-FILL DAMS

Discussion

BY MESSRS. WILLIAM P. CREAGER, AND HARRY H. HATCH

WILLIAM P. CREAGER,[42] M. AM. SOC. C. E. (by letter).[42a]—It is unfortunate that the theory of design of earth dams is not as exact as that for most engineering projects. The greatest contributing factor in this respect is the extreme variation in the character of the materials used in their construction. It is obvious, therefore, that material addition to the knowledge of the behavior of earth embankments subject to the pressure and percolation of water cannot be expected without the accumulation of an enormous amount of data regarding the characteristics of such materials in existing dams, tabulated in accordance with their reactions when subjected to accurate laboratory tests.

To be of value such tests should be strictly comparable. It is hoped that a method of testing materials for dams will be developed eventually, which will be as standard as those now available for most other building materials. The author's presentation of the testing methods used at Cobble Mountain Dam is very commendable and may well serve as a basis upon which to develop a standard method.

The writer is much interested in Equation (23), particularly in regard to the exponent of V. From this equation it is seen that the author's experiments indicate the relation between the coefficient of permeability, K, and the percentage of voids, V, to be expressed by the formula,

$$K = C\,V^{6} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (53)$$

Prior to the publication of the author's paper, the only authority for the relation between these two factors was that given by Professor Slichter,[43] as,

$$K = C\,V^{3.33} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54)$$

NOTE.—The paper by Harry H. Hatch, M. Am. Soc. C. E., was presented at the Joint Meeting of the Irrigation and Power Divisions, Yellowstone National Park, Wyoming, July 7, 1932, and published in October, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1933, by Charles H. Paul, M. Am. Soc. C. E.; February, 1933, by Joel D. Justin, M. Am. Soc. C. E.; March, 1933, by Messrs. Jeptha A. Wade, and Stanley M. Dore; April, 1933, by Messrs. D. P. Krynine, and M. M. O'Shaughnessy; and May, 1933, by Joel B. Cox, Assoc. M. Am. Soc. C. E.

[42] Buffalo, N. Y.

[42a] Received by the Secretary June 1, 1933.

[43] *Water Supply Paper 140*, U. S. Geological Survey.

and this relation has been used extensively in textbooks. There is probably some explanation for this difference, either in the method of testing or in the class of materials used.

HARRY H. HATCH,[44] M. AM. SOC. C. E. (by letter).[44a]—The purpose of tests for hydraulic-fill dams is to determine certain characteristics of materials that bear directly on the safety or the design of the structure, as mentioned throughout the paper. The standardization of tests and the development of new testing methods naturally will lead to more careful consideration of the requirements of design as pointed out by Mr. Paul, who states that,

"Core material available is limited approximately to the quantity passing the 200-mesh sieve. An excess over apparent requirements is necessary, as some of the fines will be trapped in the beaches, and it helps core control to be able to waste some of the fines"[18].

This cannot be true in all cases and should not be taken as a general principle.

The gradations of various core samples tested at Cobble Mountain (see Fig. 8), show the wide range of the percentage of material passing the 200-mesh sieve. The curves for the upper range, which show that almost 100% of the core consists of material that passes a 200-mesh sieve, are from dams in which fines have possibly been wasted during construction. In some of the dams represented in the lower ranges, the limit being only 50% of the core made up of material that passes a 200-mesh sieve, no fines were wasted during construction. No doubt there are other successful hydraulic-fill dams with cores that contain less than 50% of material finer than a 200-mesh sieve (ignoring any waste material), in which there are not enough fines to prevent excessive seepage.

The gradation tests of eight borrow-pit samples of hydraulic-fill dam material analyzed at Cobble Mountain showed that the range of variation of the material in the borrow-pits that passes a 200-mesh sieve, was about 700% between the finest and the coarsest samples.

The clay content and the distribution of particle sizes or the gradation of the material passing the 200-mesh sieve are of utmost importance.

Excess quantities of fines in the borrow-pits, greater than the requirements of the core, are not always desirable. In fact, in some cases this condition is a source of trouble and danger. Not much fine material could be wasted in the beaches. No doubt some will be trapped here and there, but in quantity it has no importance. If considerable quantities of fine materials are deliberately entrapped in the beaches, it will create a source of weakness in the structure.

For the construction of the Cobble Mountain Dam the contractor was limited to certain prescribed borrow-pits. After the first working season (1929), when the structure had been built to a height of about 60 ft, the

[44] Engr. in Chg., Cobble Mountain Reservoir, Springfield Water-Works, Springfield, Mass.

[44a] Received by the Secretary August 3, 1933.

[18] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922) pp. 1881, 1883, and 1236.

contractor located another borrow-pit, which was much to his advantage in regard to elevation, location, depth, quantity, etc. However, the material contained excessive fines, such that the percentage passing a 200-mesh sieve was approximately the same as that of the material in the core of the dam.

The contractor was told of the restrictions in the specifications, justified for various reasons, that no excess fines could be wasted below the dam. Extensive studies were made to provide special storage basins within or outside the reservoir basin for depositing these fines. After much time and thought spent on the proposition, the idea was abandoned because of its exorbitant cost and the impracticability of taking care of the excessive fines in the proposed storage basins.

In 1930, the contractor insisted on giving this material a trial and subsequently moved about 35 000 cu yd of it. Between the heel of the dam and the entrance of the diversion tunnel, a dike was constructed near the intake, to separate the supply pool or the settling basin from the Springfield (Mass.), Water Supply System during the construction of the dam. It was hoped that sufficient fines would precipitate in this settling basin, but in spite of all precautions, some were wasted below the dam, causing much trouble.

Immediate difficulty was encountered because the core section was built faster than the beach and not enough water was left in the core pool to float the pumps. The fines did not precipitate as readily in the settling basin as had been expected and, since the muddy water was used over again, it became almost impossible to operate the plant due to the fines being carried back and forth. To proceed with the operations meant increasing the depth of the core pool which, necessarily, would increase the width of the core. Furthermore, the fines in this material were such that the slopes of the beaches were flat. These factors combined to produce layers of very fine material within the beach section, such that the safety of the structure was threatened, and work had to be stopped. Remedying the situation, the dam was finished with material from the borrow-pits originally specified.

Each construction job has its own problems, and these problems often vary greatly, especially as to the feasibility of wasting excessive fines. At times the quantity of water, that is necessary for wasting fines, must be considered. It is of the most importance to know the problem; that is, to know the nature of the material available—the conditions and the restrictions governing its use—and then to design the structure accordingly.

A comparison of the gradations of core materials in Cobble Mountain Dam with those of the Miami Conservancy dams cannot be conclusive in discussing their relative consolidation and seepage without taking into consideration the respective heights of the dams and the widths of the core sections. The maximum height of the Cobble Mountain Dam is 263 ft, compared with a height of 125 ft for the Englewood Dam, the highest in the Miami Conservancy District.

Equation (30) takes into consideration the conditions after the completion of the dam. There is no dispute concerning the vertical drainage prior to completion, but no one can state definitely how much vertical drainage occurs afterward. Whatever that may be, it can properly be assumed that the drain-

age of the remainder of the excess water (or the ultimate consolidation), will take place approximately in accord with Equation (30), keeping in mind the assumptions and limitations given in its derivation.

Assuming that the two dams under consideration are built with identical materials and under the same conditions, and considering only the effect of the height and width of the core and the effective size of the material, Equation (30) becomes:

$$T = \frac{c}{H}\frac{D^2}{d^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (55)$$

in which, c is a constant. Substituting values pertaining to Cobble Mountain Dam in Equation (55),

$$T_1 = \frac{c}{263}\frac{(170)^2}{(0.009)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (56)$$

Substituting the effective size pertaining to Miami Conservancy dams in Equation (55) and adopting the Miami Conservancy design of core width at any elevation equal to the height of dam above that elevation, and with the height of the Cobble Mountain Dam,

$$T_2 = \frac{c}{263}\frac{(263)^2}{(0.002)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (57)$$

The ratio of T_2 to T_1 (Equations (57) and (56)) indicates that the Cobble Mountain Dam, with Miami Conservancy design and core material, would require 48.5 times as long for ultimate consolidation as it will under the existing conditions. The fact that the core material used in the Miami Conservancy dams showed a satisfactory rate of consolidation would not necessarily mean that the same material would show the same satisfactory consolidation if used in a dam as high as that at Cobble Mountain.

Mr. Paul offers no data pertaining to the angle of repose of materials in successful hydraulic-fill dams and their outer slopes to show that "in many cases other considerations, such as angle of repose, may require a volume of shoulder material that will eliminate the necessity of more than a passing consideration of the coefficient of friction." The outer shoulder slopes of hydraulic-fill dams are built to designed grades, usually by dikes made from sluiced beach material. During construction the dikes near the outer slopes of the Cobble Mountain Dam were built from sand and gravel excavated at the discharge end of the sluicing pipes and dumped by the drag-lines. The slope of the angle of repose of these dikes was less than 1 on $1\frac{1}{2}$, and yet the average down-stream slope of the dam was 1 on $3\frac{1}{3}$ and the average up-stream slope was 1 on 3. As emphasized in the paper, the coefficient of friction is one of the most important factors in the design of hydraulic-fill dams.

Mr. Paul's remarks in regard to prevention of tongues or layers of sand or gravel in the core and the seepage through abutments and foundations are very important. However, these are construction and control problems, and even though not exactly within the scope of this paper, the writer's remarks in another paper [45] may be repeated here, namely, (1) the engineer

[45] "Construction of the Cobble Mountain Dam," by Harry H. Hatch, M. Am. Soc. C. E., *Journal*, New England Water Works Assoc., Vol. XLVI, No. 4, pp. 327–344.

should have a thorough knowledge of the functions of the constituent parts of an hydraulic-fill dam under construction; (2) constant watch should be exercised both at the borrow-pits and at the dam to see that the quality of the structure is not affected; (3) the coarse material in the core section can be moved out and spread over the beach areas by means of drag-lines; the material which the drag-line cannot reach can be agitated and dispersed among the fines of the core section by means of jetting apparatus shown in Fig. 27; (4) in order to insure a good bond between the hydraulic-fill and the sides of the gorge particular care was taken, especially within the core area, to remove all surface cover; (5) it was soon discovered that if the sur-

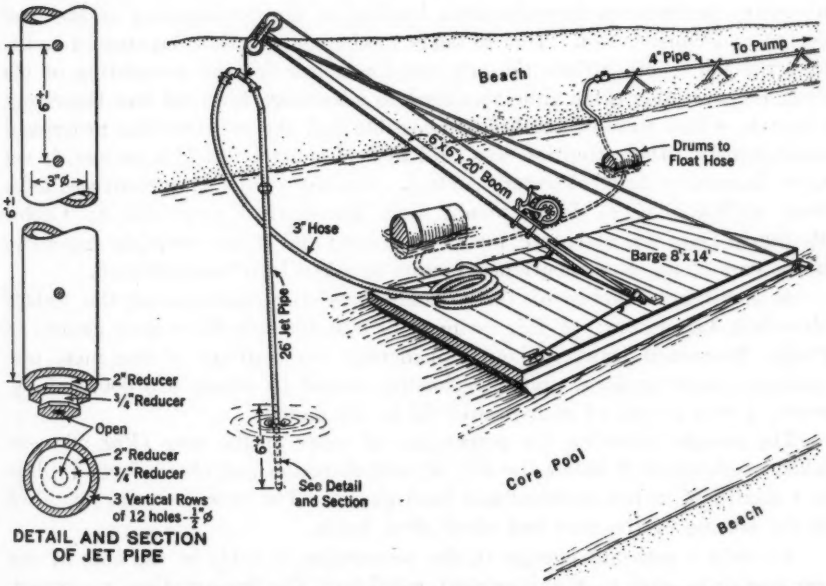

FIG. 27.—JETTING EQUIPMENT TO DISPERSE LAYERS OF COARSE MATERIAL IN THE CORE DURING CONSTRUCTION.

face cover within the core section was sluiced into the core pool, the fines would disperse all over the core area and leave the coarse material next to the sides of the gorge, creating blind drains; all such coarse deposits next to the sides were removed and, later, all the excavation was carried from the core area to the beaches; and (6) it is important to secure the required degree of water-tightness in the foundation both at the bottom and on the sides of the dam by cut-off trenches, cut-off walls, and grouting.

Answering Mr. Justin's question in regard to the costs of the tests and control, it should be stated that the research at Cobble Mountain, both in the laboratory and in the field, was undertaken with the full advantage of the practical and theoretical knowledge of several authorities on the subject. Even then, not enough definite information for all the necessary tests could be obtained. From the beginning to the end, no method dealing with the

problem at hand, developed elsewhere, or contemplated by others, was either adopted or discarded until it was given a satisfactory trial. Members of the Laboratory Staff were sent to Columbia University and Massachusetts Institute of Technology to get acquainted with the research being done by others along similar lines. Almost all the important hydraulic-fill dams under construction in the United States were visited and problems and methods of testing were studied in each case. Several sets of apparatus were purchased and used for tests, but their cost as well as the time lost in making the experiments produced nothing of value except the knowledge that the tests were not suitable for the purpose. The cost of some of the testing machines developed on the job was only a fraction of the total expense incurred for the necessary preliminary investigations leading to the development of the final design and construction. Due to delay in the construction, because of re-letting the job to other than the original contractor for the completion of the dam, the available extra time was used to advantage for field and laboratory research, which would have been impossible had the construction progressed continuously. Due attention was paid to field control which is no less, if not more, important than laboratory control. The cost of the equipment was more than $5 000. There is no doubt that the control practiced at Cobble Mountain Dam was elaborate, but with reason; and it was certainly expensive if the cost of the aforementioned factors is taken into consideration.

A maximum number of three technicians was employed at the Cobble Mountain Laboratory. A fair estimate of cost for only laboratory control at Cobble Mountain Dam, without considering the training of the men, preliminary investigations, and experiments, would be about $15 000, or, say, nearly 1 cent per cu yd of hydraulic fill in the dam.

The samples showing the percentage of voids of the core (Fig. 2) were taken at about 15 ft below the silt or mud elevation and about from 15 days to 1 month after the material had been placed. The core material deposited in the bottom of the pool had about 60% voids.

There is a constant change in the percentage of voids in the core at any one elevation, due to the increased weight as the construction progresses. Without undue expense, the percentage of voids of the core material at various elevations at the completion of the job cannot be stated exactly; but an average approximate value from observations during construction can be selected similar to that following Equation (30) of the paper, where the assumption is that the percentage of voids in the core will vary from 50% on top to 45% at the bottom, and that the approximate average is about 47.5 per cent. These values, however, will not necessarily apply to other cores.

Under a given load for a given material there is a limiting ultimate percentage of voids that remains almost constant, as expressed by Equation (24) for the local material, keeping in mind the conditions noted in the paper.[46]

With an assumed average percentage of voids and with a given specific gravity of solid material, the acting force, L, in Equation (24) at any one elevation below the top of the dam can be computed by means of graphs

[46] *Proceedings*, Am. Soc. C. E., October, 1932, p. 1335.

similar to Fig. 28. With the assumed and final percentage of voids, the approximate time necessary for ultimate consolidation can be computed by means of Equation (30) similarly to the numerical problem immediately following that formula.

Mr. Justin's reference to Mr. Hazen's investigation of the Calaveras Dam evidently applies to the latter's statement:[47]

"Without attempting precision of statement, it is the writer's judgment that if core material like that at Calaveras could be consolidated to 40% of voids, it would be sufficiently stable to do its share toward resisting the pressures that come on the dam without tending to disruption. Perhaps this degree of stability would be reached before consolidation had proceeded as far as 40% of voids. There is a middle ground of uncertainty. Material with 50% of voids is still unstable; and between this limit and 40% one cannot be certain just where to draw the line."

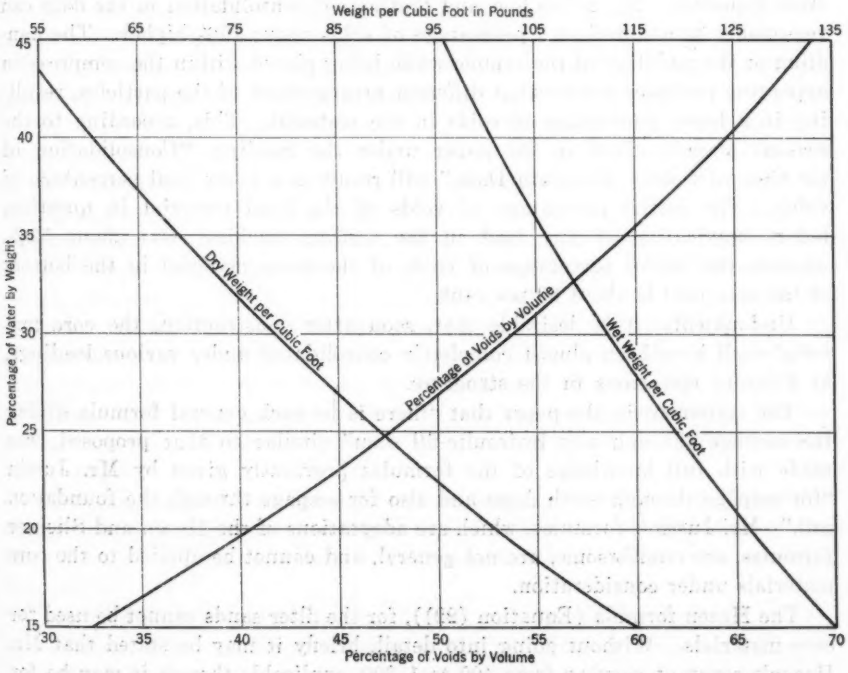

FIG. 28.—CURVES SHOWING RELATION OF WATER CONTENT TO PERCENTAGE OF VOIDS, AND WEIGHT PER CUBIC FOOT FOR SATURATED MATERIALS. SPECIFIC GRAVITY OF SOIL PARTICLES = 2.72.

Mr. Hazen did not state, nor does Mr. Justin state, at what elevation in the Calaveras Dam, or under what given load, the 40% of voids is desirable; nor are there any data to show the ultimate rate of consolidation under various loadings, or at various heights of the dam, for the Calaveras core material.

[47] "Hydraulic-Fill Dams," by the late Allen Hazen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–1920), p. 1720.

The twenty-eight core samples tested at Cobble Mountain under a pressure of 200 lb per sq in. showed a wide range in percentage of voids at ultimate consolidation. The minimum was 30% and the maximum, 41.5%, depending on particle sizes and other characteristics of the same materials. The percentage of voids at ultimate consolidation cannot be uniform throughout the core, since naturally it is a minimum at the bottom and increases gradually, depending on the uniformity of the material, toward the top of the dam.

The local material for which Equation (24) was obtained, has an average of about 38% of voids at ultimate consolidation under a loading of 200 lb per sq in., corresponding to the maximum pressure at Cobble Mountain Dam. At a pressure of 100 lb the same material will have 40%, at 50 lb, about 41.7%, and at 25 lb, about 43.5% of voids for ultimate consolidation. There is every reason to believe that the percentage-of-voids curve for the core, as obtained from Equation (24), is too low, and that actual consolidation in the dam can reasonably be assumed at a percentage of voids about 10% higher. The handling or the puddling of the sample while being placed within the compression apparatus produces a somewhat different arrangement of the particles, resulting in a lower percentage of voids in the material. This, according to the reasons already given in the paper under the heading, "Consolidation of the Core of Cobble Mountain Dam," will result in a lower final percentage of voids. The initial percentage of voids of the local material in question, before application of any load in the testing machine, was about 53%, whereas the initial percentage of voids of the same material in the bottom of the core pool is about 60 per cent.

Undoubtedly, it is desirable that, soon after construction, the core material shall have been almost completely consolidated under various loadings, at different elevations in the structure.

The statement in the paper that "there is no such general formula giving the seepage through any hydraulic-fill dam" similar to that proposed, was made with full knowledge of the formulas previously given by Mr. Justin "for seepage through earth dams and also for seepage through the foundation soil." Mr. Justin's formulas, which are adaptations of the Hazen and Slichter formulas, are cumbersome, are not general, and cannot be applied to the core materials under consideration.

The Hazen formula (Equation (22)), for the filter sands cannot be used for core materials. Without going into detail, briefly it may be stated that Mr. Hazen's constant varying from 400 to 1 200, applicable though it may be for filter sands, cannot be applied to core materials. The Hazen formula does not have any factor to take care of the variation in the percentage of voids of the material. It may be claimed that filter sands are more or less alike, and perhaps that they have a uniform percentage of voids; but the same argument cannot be applied to the core materials of hydraulic-fill dams in which there is such a variation in the percentage of voids. Furthermore, the effective size and the 60% size of filter sands cannot be compared with the respective sizes of core materials in hydraulic-fill dams. Here, again, the conditions are different.

Professor Slichter refers to his own formula for determining the flow of water through a column of sand,[48] but he does not state how he devised it. In the absence of any definite information, and in view of the fact that his discussion refers to sands and to the Hazen formula, the writer believes that Professor Slichter's formula will not aply to core materials.

The formula may be valid for sands within the range of sizes used in the Hazen experiments, but it is doubtful whether it will be reliable for fine particles of about 0.01 mm, as given in his tabulation of "transmission constants."[49] His results are probably due to theoretical computations, and his experiments, if any, did not cover the lower ranges given in his table. It is doubtful whether, about 1900, gradations were made as fine as 0.01 mm. The "transmission constants" tabulated by Professor Slichter, indicate that the value of K varies approximately as $V^{2.3}$, whereas Cobble Mountain experiments for core materials (Fig. 15), show that K varied as $V^{5.40}$ to $V^{7.2}$, or approximately as V^6, which is the average value adopted in Equation (23).

In the preceding three paragraphs will be found the reasons for the differences in Equations (63) and (64), as pointed out by Mr. Creager.

Mr. Wade's discussion of the theory of the hydrometer is a valuable contribution on the subject. It would be desirable to have complete knowledge of the underlying principles of the hydrometer; to know exactly what is taking place during the test; why the particles do not settle according to theoretical calculations; the effect of the Brownian movement; whether there is any vortex motion; and what the real reasons are for the variation in results obtained by the hydrometer compared with those by more accurate methods. These factors are not well known yet, and any possible attempt to discuss them theoretically would have been outside the scope of the paper. More space has been given to the hydrometer than is warranted, when it is considered that the hydrometer method is only one among several for making gradation tests, and that the latter is only one of the many subjects treated in the paper.

Certain principles were given to explain reasons for the method of conducting the tests, and certain probable causes were enumerated for some of the variations of the obtained results compared with those by other more accurate methods.

The adoption of the hydrometer method, with reasonably accurate results, as described, will depend perhaps more on the simplicity and almost negligible cost of the instrument, and the speed of the tests, rather than an immediate thorough knowledge of the underlying theory.

Quite properly, Mr. Wade points out the possibility of confusion in interpreting the statement pertaining to concentration at the center of buoyancy. This would have been clear perhaps if it had been stated that the resultant buoyant forces of the uneven concentrations at various elevations in the suspension acting on the hydrometer are observed arbitrarily at its center of buoyancy.

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[48] *Water Supply Paper No. 67*, U. S. Geological Survey, p. 24.

[49] *Loc. cit., No. 140*, U. S. Geological Survey.

The assumption that the center of buoyancy is the critical point to which to measure the depth below the water surface, seemed logical and gave satisfactory results, even though there was "a slight error in the assumption," according to Mr. Wade.

By checking the readings of the hydrometer with various predetermined quantities of grammes of soil per liter of water, or with solutions of known concentration, as recommended under the heading "The Hydrometer," Mr. Wade's problem with respect to variation in the specific gravity of the material in suspension can be avoided. It is doubtful whether there will be any change in the specific gravity of a core material, during the process of settlement, provided this material is nearly free of organic matter, and provided the particles of soil in the sample are of a practically uniform specific gravity.

The dispersing agents used at Cobble Mountain were mostly ammonia, sodium oxalate, and sodium silicate. The proper quantity to be used for a given soil sample can be obtained by trial beginning with a small quantity and increasing it gradually until the right proportion is reached.

The hydrometer and the thermometer readings can be estimated to the nearest one-tenth, and no doubt the nearest one-half gramme or one-half degree would suffice. Even though the corrections made on the hydrometer readings in Table 3 could have been neglected, they were inserted to show the arrangement of observations and the method of computation.

At 68°F the viscosity of water was assumed to be 0.01005 poises, or 1.005 centipoises. It is desirable, of course, to have water in the graduate with a temperature of 67°F in order to avoid corrections entirely, which, otherwise, should be made according to Equation (10). The corrections can easily be obtained from a graph of this equation.

In addition to the gradation, Mr. Dore suggests making determinations of grain shapes, their classifications, their surface and other characteristics. This would tend toward a high degree of academic refinement; but, no doubt, such studies made in a research laboratory would be of value in construction work if and when they can be readily applied in a practical manner. It is doubtful whether they can be included among the tests to be made in the field laboratory during construction. It should be remembered that laboratory and field control must supply data for quick and decisive action because of extreme variation of the characteristics of the material on hand and the urgent necessity of speed.

The surface characteristics of the grains, and their hardness are of great importance, and periodic examinations by microscope and otherwise were run at Cobble Mountain, with the result that important changes were made during construction. It is obvious that the information will be more complete if various characteristics and properties of the material in question could be presented in an accepted and compact form. It is to be hoped that investigation of the coefficient of friction of earth dam materials will continue until sufficient data are obtained on this subject.

The merits and the limitations of the three methods of testing for percolation, as finally adopted at Cobble Mountain Dam, were stated in the paper.

Each has its own advantages, depending on the available time for tests and consideration of cost. It was fortunate that at Cobble Mountain each of these methods was available and was used, because some of the results obtained and conclusions reached are due to their collective use.

In regard to the vertical and horizontal capillary tube methods, it may be stated that the dry sample of material placed in a glass tube cannot be converted into a material similar to that in the core of a dam. The dry sample often contains small lumps of material that are not broken up even with reasonable care in its preparation. The lump content of samples is not the same, and it does not follow any rule. The presence of lumps accelerates the capillary rise of water in the sample in proportion to their size and number.

These methods are more or less approximate. The vertical capillary method requires a series of master curves, prepared carefully for similar materials, and by elaborate methods. The horizontal capillary formula contains a constant, depending on the percentage of voids and the character of soils, which can only be obtained after the coefficient of permeability of that one material has been determined carefully by numerous experiments conducted with reliable methods. The fact that similar approximate values or results could be obtained more easily and readily from Equation (23) was the reason for discarding them without prejudicing their value for testing rolled earth dam material.

It was not intended that Equation (23) should replace the actual tests and direct measurement of the coefficient of permeability of the core material, but that it should be used only for estimating, approximately, the coefficient of permeability. This formula is the best tool available for the purpose. In order to be of more general use in its application to the wide range of core materials, it requires a factor, expressed in terms of the area, perhaps below the curve, between size limits of 60 and 10%, or (which is better) as nearly as possible between the full limits of 100 and 0 per cent.

The relation between the coefficient of permeability, K, and the effective size, d, as shown in Equation (23) was obtained from the results of thirty-nine complete experiments using two different sets of percolation apparatus. A study of the plottings of Fig. 29 will show that for the large percolator the range of the coefficients is from 84 to 480 and the general average for the 18 tests is 176. The compression apparatus gives a range of coefficients from 30 to 311, or a general average of 106 for 21 tests. The average of the values obtained from the two sets of apparatus is 141.

The following test results were questioned during the experiments:

Sample No.	Value of K	Apparatus used
$P1$.....	228.....	Standard compression
$P2$.....	311.....	Standard compression
$P4$.....	480.....	Large percolator

Omitting these questionable data, the range of the coefficients is from 84 to 278 for the large percolator and the general average for the reliable 17 tests is 157. Similarly, the range of coefficients is from 30 to 163, for the com-

pression apparatus, or a general average of 89 for 19 tests. The average of the values obtained from the two results is 123.

The adoption of K equals 150 instead of 141 or 123 is due to preference given the large percolator, which may be more reliable for this test in view of the fact that its sample was more than 400 times larger than that used for

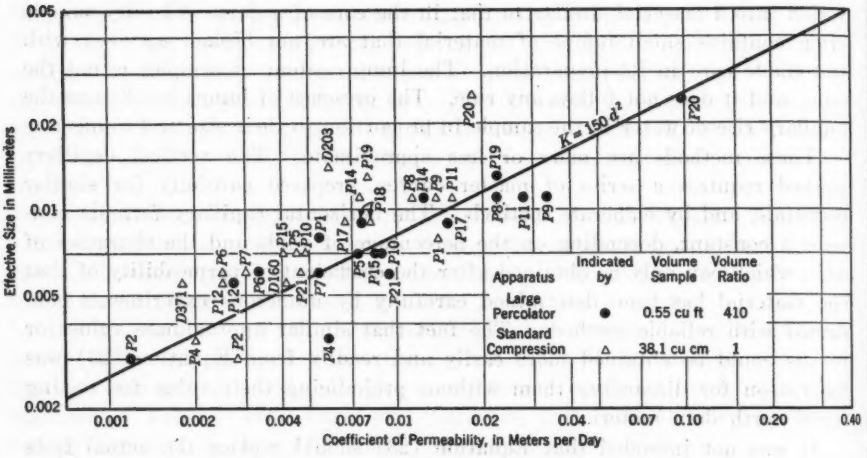

FIG. 29.—RELATION BETWEEN EFFECTIVE SIZE AND COEFFICIENT OF PERMEABILITY AT 45% VOIDS AND 50 DEGREES FAHRENHEIT.

the compression test. Equations (23), (30), and (43) afford valuable information if used with discretion, keeping in mind their restrictions or limits as pointed out in the paper.

Due to the fact that the gradation results obtained by the hydrometer and elutriator methods are almost alike, Professor Krynine raises the question as to why the elutriator needs to be used for future tests of the same character. At Cobble Mountain, hydrometer results were often checked by the elutriator, thus making certain that the hydrometer results were reasonably accurate. If the hydrometer runs at a construction laboratory can be checked periodically by a more accurate method, whether the elutriator or some other, there is no reason to invest money in the apparatus and devote time to making elutriator tests.

Attention is called to some facts in regard to the arbitrary assumption relating to the center of buoyancy of the hydrometer bulb, which, when fully studied and analyzed, may clarify further the reasons of the discrepancy (especially in the lower limits), between the results of the hydrometer and a more accurate method.

The curves in Fig. 10 were plotted only within the limits of actual experiments, and the corresponding equations are likewise limited. It seems logical, however, that as the vertical pressure approaches zero the coefficient of friction should approach infinity.

It is regretted that Professor Krynine did not reproduce from German and Russian books on seepage, a formula, if any, similar to Equation (43), that would give the seepage through a hydraulic-fill dam. It would be interesting to compare such a formula with Equation (43) of this paper. Equation (43) was derived without any knowledge of the works referred to by Professor Krynine.

Mr. O'Shaughnessy states that "the core material is of particularly thin section for this type of dam." This is true for Cobble Mountain Dam as compared with others in existence. The thin core section has given good satisfaction, however; the reservoir has been full for some time and the seepage through the structure has been negligible.

The large weep-holes in the toe dam will scarcely affect the resisting strength of the structure, and there is no danger of their being plugged by fine material, resulting in hydrostatic pressure back of the toe dam.

According to Mr. Maitland C. Dease, to whose report Mr. Cox refers, considerable work has been done at the Alexander Dam along lines suggested by Professor Terzaghi for tests on clays. These methods were not given much consideration at Cobble Mountain, because the problem was not concerned with clays. The "scope and intent" of the testing program at Cobble Mountain Dam was practical, and was as complete as possible.

Highly theoretical tests made in research laboratories, are valuable, but in the construction of an hydraulic-fill dam the most important criterion is effective field control, guided by laboratory tests. The laboratory investigations for this purpose should bear immediate results, which, in turn, should become useful for immediate application. The interpretation of the laboratory and field investigations is of utmost importance. Neither elaborate machinery, long-period tests, nor complicated equations are necessary in order to detect danger signals pertaining to the safety of the structure. Simple terms, common sense, and practical judgment in interpreting tests are more valuable than tests requiring months, which produce complex equations not readily understood. Lack of practical interpretation often leads to disastrous results, no matter what the testing program may have been.

The aim at Cobble Mountain was to simplify the testing methods and to make frequent tests so that their results could serve as guides during the construction. Changes were possible immediately, without hindrance to the progress of construction, and yet with time to produce the required stability in the structure.

A study of Mr. Dease's full report reveals no information in regard to the number of tests for the coefficient of friction at Alexander Dam comparable with about 550 tests made at Cobble Mountain Dam. The report indicates that during a month only six core samples and only three beach samples were tested, whereas at Cobble Mountain Dam an average of more than 100 core samples alone were tested each month. There is no evidence in the report to substantiate Mr. Cox's statement that the testing program at Alexander Dam "was considerably more extensive than that at Cobble Mountain, or than has heretofore been attempted in connection with hydraulic-fill construction."

Shearing resistance of beach material is of interest; no studies of the effects of cohesion and friction in their combined relation to shear were needed at Cobble Mountain, because the beaches were composed of almost cohesionless sand and gravel.

Whereas a material as fine as that used in the Alexander Dam might require the pycnometer method for accurate determinations of the specific gravity, the simple displacement method gave entirely satisfactory results with the coarser and less cohesive material used in the core of the Cobble Mountain Dam. On the other hand, considering that at Alexander Dam "the specific gravity of the separate grains of a representative sample varied from 5.2 to 2.2" (a variation of 236%), it is questionable if it was necessary to use such refined methods of determining the specific gravity, because the "simple displacement methods were frequently in error as much as 15 per cent."

The voids in the large percolator described in the paper were obtained with sufficient accuracy by simple moisture content determinations, and the variation within the apparatus was measured and found to be insignificant. The stand-pipe permeameter is admittedly difficult to load with some types of material and under some conditions and hence it is limited in its application, but where it is suitable, it is simple and quick.

The term, "percentage of voids," is preferred to that of "voids ratio," because of its simplicity, and because of the fact that it has been used for many years and is readily understood by practising engineers.

The simplicity of Equations (21) and (24) compared with Equations (46) and (47), speaks for itself especially when "there is little actual difference over a moderate range of voids-ratio." Both sets of formulas are empirical, and are applicable to particular materials. The ultimate aim might be to develop a set of formulas sufficiently general and complete to cover all materials and containing commonly accepted terms. Until such formulas are available the most simple, rather than the most technically precise, formulas would seem desirable for working purposes.

A study of the curves in Fig. 22 shows that they can be expressed by the general formula given in Equation (21), but that the exponent, m, will be much higher than 7.2 as given in Table 5.

The permeability coefficients in Table 5, evidently do not take into consideration the temperature factor, which is important.

The permeability coefficient given for the Cobble Mountain Dam material is not correct, and, furthermore the units of centimeter per minute and meter per day are not the same. Chemical content and mineralogy were not serious problems at Cobble Mountain Dam, but their importance as pointed out seems evident, and of considerable interest.

The time required for reasonably accurate work by microscopic examination makes this more properly a research, rather than a field laboratory, test. Due to the relatively uniform specific gravity of Cobble Mountain Dam material the difficulty with variations was not encountered; there was no indication of excessive hydration. The other factors are undoubtedly present and their influence depends on the character of the material. These factors must

be estimated for material from any given locality, since they are not susceptible at present to complete, or exact, analysis. The discrepancy in the gradation curves obtained by hydrometer and microscope, as shown in Fig. 21, is interesting, if these curves are the results of numerous sets of experiments.

The discussion of shearing tests on clays and clay-like soils is of considerable interest and possible value in soil studies. Its importance with direct relation to an hydraulic-fill dam is not so evident. The fundamental concept

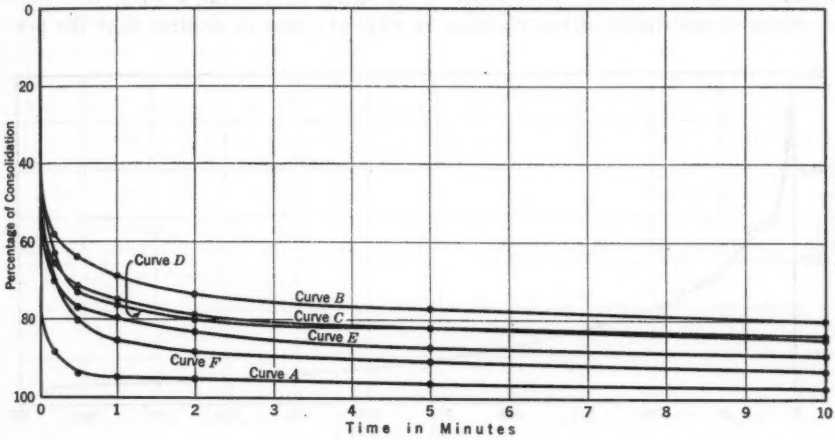

FIG. 30.—CURVES SHOWING RATE OF CONSOLIDATION OF A COBBLE MOUNTAIN DAM CORE SAMPLE.

of a true hydraulic-fill dam is one in which the beach section is of a sufficiently coarse material to permit almost immediate drainage and consolidation of itself, such as to allow suitable drainage facilities for the core. The safety of an hydraulic-fill dam being dependent on the coefficient of friction, or shearing resistance of the beach section, the application of shearing tests to this coarser material would seem more important and applicable.

TABLE 6.—IDENTITY AND DESCRIPTION OF CURVES IN FIG. 30

Curve in Fig. 30	Pressure, in pounds per square inch	Percentage of Voids		Time, in hours, for 100% consolidation
		Initial	Final	
A	8.6	49.6	45.0	20
B	13.0	45.0	44.4	19
C	25.0	44.4	43.2	23
D	50.7	43.2	41.6	21
E	101.5	41.6	39.6	17
F	199.5	39.6	37.6	22

Fig. 30 (in which the curves are similar to those of twenty-six other sets of consolidation experiments made at Cobble Mountain) and Table 6 will substantiate the statement that "the rate of consolidation of material under a given constant load, shortly after the applicaion of the load, becomes almost uniform." For example, Table 6 shows that 100% consolidation was reached

for Curve *A* in 20 hr, whereas, by Fig. 30, the sample had 97% consolidation in 10 min. Likewise, Curve *B* reached 100% consolidation in 19 hr (Table 6), whereas the sample material reached 80% consolidation in 10 min. This rapid consolidation undoubtedly is due to the physical characteristics of the Cobble Mountain material. Perhaps the conditions would be different for clays, but the same tendency is evident in Fig. 31, which was replotted from Fig. 23 for Alexander Dam material. In the absence of any definite data in regard to the initial and final voids of the sample material, assume that they conform to the limits of the plotting in Fig. 31; that is, assume that the per-

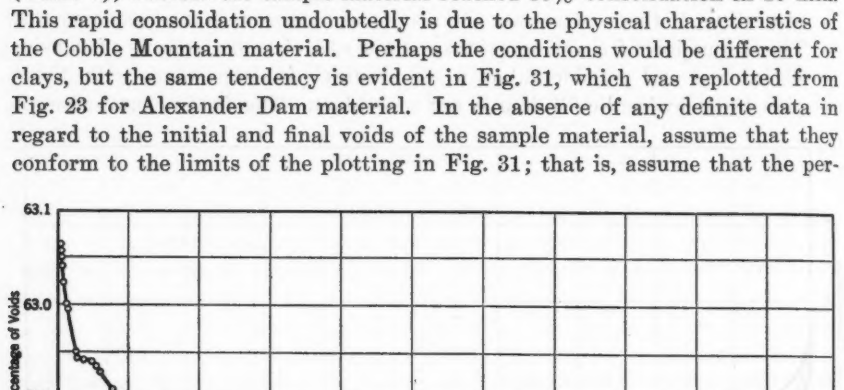

FIG. 31.—COMPRESSION UNDER A STEADY APPLICATION OF LOAD (RE-PLOTTED FROM FIG. 23).

centage of voids was 63.065% when the load was first applied and 62.767%, after the material had become completely consolidated. The consolidation is very rapid at the beginning, and for almost the last half of the total time required for the ultimate consolidation of this material there is no reason why the rate of consolidation could not be assumed as "almost uniform."

It was clearly stated that the consolidation study was only "an attempt * * * to estimate roughly the time necessary for approximate consolidation." So many factors enter into the computation—there are so many assumptions and uncertainties—that it is not worth while to go into too much refinement. The purpose of the study was to determine whether the approximate consolidation would take, say, 2 or 20 years; even if the results obtained for the approximate consolidation vary from those of the actual, say, 100%, or, from 2 to 4 years, the study serves the purpose for which it was made.

Mr. Cox has computed the time for the consolidation of the Cobble Mountain Dam by Professor Gilboy's method, and the results do not agree with those of Equation (30); but the difference between the results by the two methods is due only to errors in Mr. Cox's computation, or assumed dimensions, rather than to the assumptions, or to the derivation of Equation (30).

"Taking an elliptical cylinder (semi-major axis, 5 930 cm, and semi-minor axis, 4 780 cm) that has an area roughly equivalent to that of the core, and a ratio of diameters about that of height to one-half the width of the actual

core," as recommended by Mr. Cox, is erroneous and cannot apply to the core of the Cobble Mountain Dam. Far from being "roughly equivalent," the area of an elliptical cylinder with the given dimensions is approximately five times that of the dam core (see Fig. 17). In order to have "a ratio of diameters about that of height to one-half the width of the actual core," the semi-minor axis should have been about one-half the value assumed by Mr. Cox. Applying the dimensions used by Mr. Cox to the Cobble Mountain Dam, or assuming twice the width of the actual core, and without going into any further details, it is seen, in Equation (30), that the time of consolidation varies as the square of the core width; or, in this case, it would be increased four times that given in connection with Equation (30) in the original paper which agrees with the tabular values given by Mr. Cox. It is interesting to note that results by Equation (30) agree so well with those obtained by more complicated and elaborate methods.

The seven reasons given to warn against deceptive conclusions, were pointed out after serious and repeated practical observations. They are still valid, at least for the Cobble Mountain materials, notwithstanding the objections offered by Mr. Cox.

It is not possible, with a few cubic centimeters of material enclosed in a polished brass cylinder, to duplicate the changing conditions and the characteristics of 300 000 cu yd of core material in a dam 263 ft high. There is a practical limit to the possibility of determining the reactions in a large core by means of theoretical laboratory investigations conducted under ideal conditions and with small test samples that are not necessarily representative.

Undoubtedly, much additional research will have to be done, and perhaps new tests and testing methods will be devised in connection with hydraulic-fill construction; but the time has come when engineers connected with this type of construction should get together and co-ordinate the various methods of successful testing and to classify data gathered from successful structures. It is time that engineers should talk the same language and express themselves in the same terms and definitions so that differences in results can be distinguished readily by practising engineers who are not concerned with abstruse theories, the application of which to the problem on hand may yet be somewhat questionable. There is no reason why, until something better is offered, the consensus of opinions and facts compiled by actual experience should not serve as a guide for this type of construction. Establishment of standard tests and testing methods in no way implies that further research should be stopped; on the contrary, the present chaotic condition will be clarified, and much duplication of work will be avoided. Time thus saved can be used for research in the other investigations necessary to a solution of the many features of the problem.

Throughout the paper readers were cautioned against accepting any conclusion as final, and against applying it to another locality in which the materials were different. For a quick decision, or for estimating the probable results during construction, the utility of data obtained from grain sizes and their gradation is paramount until some better and quicker method can be developed. With thousands of yards of material daily going

into the structure, there must be some practical means by which the engineer responsible for its safety can readily determine the course to be followed in the control of the construction, if conditions so require. It would be out of the question to stop the construction for weeks at a time in order to determine what should be done.

The writer feels grateful to the discussers, even though some of the views expressed were contrary to those in the paper. After all, however, the difference in opinion will help to bring out the fundamental principles of the subject-matter more definitely than ready acquiescence.

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DISCUSSIONS

A PROBLEM OF SOIL IN TRANSPORTATION IN THE COLORADO RIVER

Discussion

BY S. L. ROTHERY, M. AM. SOC. C. E.

S. L. ROTHERY[51], M. AM. SOC. C. E. (by letter).[51a]—The questions cited by Mr. Grunsky, as to the physical characteristics of the Colorado River silts, need to be answered by research as a first step in attacking the problem, the magnitude of which, he states, is unquestioned. Acceptance of a difficulty without seeking its analysis is contrary to engineering principles. Suggested answers to unexplained riddles often present themselves to the questioning mind, but the light of research is wanted in order to reject or verify suggestion.

The riddle of the turbid flow problem appears to be: Why do the suspended silts maintain general uniform distribution through a flowing cross-section, the charge or percentage of load remaining practically unchanged to any distant down-stream point?

A suggested answer may be reasoned from the studies before referred to[22] contrasting the shapes of silt particles to those of sand. Professor Terzaghi states that even a high-powered miscroscope does not discern the constituents of "micro-mud" (fine silts), but claims that their scale-like property is supported by many proofs, some of which are stated in the aforementioned studies. It is somewhat disappointing that none of the discussers gave new light on the physical difference between silts and sands, particularly as regards their carriage in stream flow and their characteristics of deposition in quiescent water. Partial settlement will be of little help in removing the turbid appearance in the first years following the storage behind the Boulder Dam; however, the relative volume of finest silts remaining in the 250 miles of river channel down stream, is so small in comparison to the present annual suspended load, that the question is again raised as to whether great desilting basins are really required.

NOTE.—The paper by S. L. Rothery, M. Am. Soc. C. E., was published in December, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1933, by Messrs. C. E. Grunsky, Ivan E. Houk, and H. M. Rouse; May, 1933, by Messrs. William T. Collings, Jr., R. F. Walter, Harry F. Blaney, and Calvin V. Davis and Dirk A. Dedel; and August, 1933, by Messrs. Thaddeus Merriman and J. C. Allison.

[51] Care, Beattie Gold Mines, Ltd., Noranda, Que., Canada.

[51a] Received by the Secretary July 7, 1933.

[22] *Engineering News-Record*, Vol. 95, p. 912.

In explanation of the small remaining volumes, it must be remembered that the origin of the fine silts is above the Boulder Reservoir, a function of which is to trap the present annual quantities of 90 000 to 110 000 acre-ft of suspended silts. The Imperial Diversion will be 293 miles below the reservoir, and the river-channel area in this length is about 31 000 acres, according to U. S. Geological Survey Topographic Sheets. The effect of raising the water surface at the diversion structure and also at Parker Dam, will be to subject only a part of this length and consequent area to the degrading process, before the ultimate new gradient is attained, say, to 250 miles and 25 000 acres. Should the ultimate maximum recession be 25 ft, the mean at 12½ ft would give a product of 312 500 acre-ft to be removed in, say, 10 years, one-third (34%) of it being the "fines". Then, again, since only one-fourth of the annual flow is diverted, the future turbid problem appears to become so greatly reduced that it seems reasonable to expect a noticeable improvement in the clarity of the flow soon after the Boulder Reservoir is in operation.

A careful investigation of the silts in the flowing canals was undertaken by the Department of Engineering of the State of California in 1918–19.[52] Quoting from this report:

"By far the greater part of the total silt in suspension in the Colorado River and the irrigation canals is fine material that passes the 280-mesh per inch screen, * * * this material was in all cases found to be equally distributed throughout the vertical for all velocities under which tests were made, including mean velocities of less than one-half foot per second in small ditches. * * * The silts of the Colorado appear to be comparable to those of the Nile in degree of fineness, but they are much finer than those of the rivers of India."

Silt research on the Nile,[53] showed that in the lowest one-third depth, where the river was 26 ft deep, about one-half the coarse (that collected on a No. 100 sieve) and of the fine (that collected on a No. 200 sieve) sands, were located. It was certainly not the writer's intention to infer that there was a definite line of demarcation between the silts and sands in the turbulent floods of the Lower Colorado River; but, in the expected future regulated flows, when the maximum discharges will be from one-half to one-third their former volumes, and where the stream regimen can be improved to avoid turbulent approach, most of the sands in transportation will be near to, and along, the bed, which will be about 12 ft below the water surface. If a structure divides the water such that the sands in the lower one-third of the stream section will be carried on through under-sluices, those grains diverted to the canal with the upper two-thirds, will amount to only a minor percentage of the entire sand load, and this percentage could be eliminated in sand-trap structures at appropriate wasteways.

The tributary streams and washes in the 293 miles will occasionally be a source of additional sand, and perhaps some fine silts, to the river. However, the discharges from cloudburst rains on these tributary water-sheds are far

[52] Seventh Biennial Rept., Dept. of Eng., State of California, pp. 117–120.
[53] *Minutes of Proceedings*, Inst. C. E., Vol. CCXVI, pp. 205–206.

from being continuous inflows, and it is believed that the contributed water is only the equivalent of a few days' average flow of the main river.

Mr. Rouse believes that the desilting basins should be built in addition to the desanding works, and suggests certain locations other than at the diversion structure. In this consideration (and always having in mind the time of detention of the flow to precipitate the fine silts referred to in the foregoing quotation, which from years of experience on the delta is emphatically endorsed), the writer offers the following plan as best adapted to obtain a maximum clarity of water for the present ranchers and the beneficial effect of silty flow for the new lands to be reclaimed on the East Mesa:

(*a*) Obtain the best possible desanding by under-sluices in the diversion structure, and perhaps a sand-trap structure across the canal not too far distant from the main structure, making no attempt to precipitate the suspended silts;

(*b*) Construct separate stilling-basins for the Yuma Project, to desilt 1 500 sec-ft, at a suitable location to the head of the Yuma Main Canal and where the sludge can be sluiced to the river, probably somewhere in the vicinity of Station 250 on the survey; and,

(*c*) Construct separate stilling-basins for the present irrigated lands of Imperial Valley, to desilt 5 000 (perhaps 7 600) sec-ft at about the west edge of the East Mesa, say at Station 2 990, sluicing along an obtained right of way to the Alamo Channel.

Then, whatever degree of desilting is attained by the type of basins devised, the problem is much simplified, due to the lesser volumes, than if one were seeking to detain 10 000 to 15 000 sec-ft, which has a velocity of 4½ ft per sec; and a greater efficiency of desiltation could be accomplished at a minimum of construction and of daily operation costs. Furthermore, the long main canals crossing the porous soils to the East Mesa and on the Mesa, would receive the benefits of silty flow to help minimize percolation losses and contiguous areas of seeped lands.

It is true that an extra few hundred second-feet of sluicing water must be conveyed to the locations. The objection to the sludge going to Salton Sea is of growing lesser importance, since it has been argued that the suspended load is a decreasing one yearly and there is a relatively small total volume of the "fines" to be removed below the Boulder Reservoir.

Should the present lands of Coachella demand clear water, a smaller pondage could be provided near Salton, Calif., sluicing to the Salton Sea at about Station 4 200 on the Coachella Canal where the velocity is only 2.7 ft per sec.

The writer admits, and expressly stated in the paper, that the figures used to present a viewpoint of the bed load, were necessarily assumptions. His claim to being conservative in the volumes deduced can be re-stated, since in one instance a scour depth of only 9 ft was used, and, in the second instance, only 50% of the 300 miles in length were taken to allow for "deeps" and alternate "shoals". Both instances produced a removal (during a few weeks' time) of soil greater than the estimated yearly suspended load.

When excavating for the foundation of Boulder Dam, where the low-river water surface is at Elevation 645, a piece of sawed 2 by 6-in. board was found firmly embedded in gravel at approximately Elevation 595. The difference of 50 ft is indicative of deep scour. Reference has been made to the deep scour at Yuma,[13] and the writer recorded[54] a scoured depth of 20 ft on the delta on a rising flood, when the discharge was only 60 000 sec-ft and when the metering cableway was swept away in the flood. This flood increased to 110 000 sec-ft without any further raising in elevation of the water surface, indicating a still greater depth of scour. It must also be conceded that the distance of 300 miles is shorter than that in which the scoured material has been excavated and removed by the gradual rise of the spring flood.

The writer is not alone in pointing to the indications of this voluminous bed load. In 1930, Thaddeus Merriman, M. Am. Soc. C. E., stated[55]:

"In any event, indications are that the volume of this [bed] load, when determined, will prove to be substantially greater than the suspended load which is now known within reasonable limits."

Mr. Merriman now confirms his previous conclusion. Mr. Blaney also makes a reference to the fact that there has been lack of recognition of the greatness of the traveling bed load. Such statements are of great value and will help engineers to realize the magnitude and the problem of the bed load, which the writer has endeavored to feature a number of times since 1921.

It was not the writer's intention that his paper should have been taken as a criticism of the tentative plans of the Bureau of Reclamation. During years of experience on the delta, he observed instances of scour, of deposition, and of suddenly changed river courses that seemed incredible to account for, until familiarity with the region and its disemboguing mud torrent provided a logical reasoning for what at first were surprises. He is convinced that the valuable estimates of soil transported as "silt in suspension", comprise only a part of the story. The silt problem of the Lower Colorado River has always been of absorbing interest, which has been responsible for much personal research and deduction. In submitting this paper, spontaneously written in a few days, the writer now believes that his presentation as to (*a*) a large bed load, and as to (*b*) the feature that its removal can be effected with the usual under-sluices while it is practically impossible to remove the turbid condition of the flow for great volumes of moving water, stress the fundamental principles of the problem.

Few engineers experienced on the Lower Colorado River have voiced their findings of silt characteristics. Realizing the importance of the subject for determination by specific research, in the "Synopsis" of the paper, the writer invited the publication of such research and the pooling of available knowledge. It is refreshing to know of the important investigations now in progress, referred to by Mr. Walters.

[13] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1142.
[54] *Loc. cit.*, Vol. LXXXVI (1923), p. 1417.
[55] *Loc. cit.*, Vol. 94 (1930), p. 1144.

The psychological effect of expensive desilting basins is promissory of clear water to the canal systems; the writer had endeavored to show that this result is economically unattainable, since turbidity is produced by the "fines" (silts passing a 200-mesh sieve, and not susceptible to rapid deposition). The excessive "muddiness" of the water is the reason for the so-called desilting basins; yet only a fractional part, if any, of these excessive volumes of "fines" would be removed by the basins.

The reduced silt content, referred to by Mr. Blaney as about 50%, at the Laguna Dam Diversion to the Yuma Project, is suggestive that the multiple-basin plan will be equally effective for a diversion ten times greater. This may or may not be true. As the "fines" are not removed by the retarding basins, it may be preferable not to remove the coarser silts in carriage in suspension, in order to help guard against producing the objectionable sticky cultivating soil developing on the Yuma Project. In comparing efficiencies of proposed desanding basins with those of suggested under-sluices for eliminating sand in the approaching water cross-section, it would seem important and desirable to consider this precaution, the ideal requirement being to eliminate such sufficient part of the traveling sand as will suffice to prevent sand blocks occurring in the Main Canal. This is probably a matter of practical judgment from noting similar conditions and treatments on other great world projects.

The Sarda Canal, in Northern India, designed to carry 9 500 sec-ft, takes off from the Sarda River which "carries immense quantities of coarse silt during the flood season, and it was feared that in spite of all precautions, a considerable quantity would pass into the canal."[56]

The Sarda diversion structure is a side offtake "consisting of 16 bays of 20-foot spans. There is a masonry sill curved up stream to give a stream-line entry and thus avoid eddies which might cause silt to be picked up from below." The contiguous river sluicing sill or apron is at a depth of 6.75 ft below the masonry sill. Behind the sill are rising steel gates, which are usually lowered to permit the passage of top water over them into the canal. It is reported[56] that the canal head is working admirably; it excludes all heavy silt, and only the fine silt in suspension, which is useful for forming water-tight berms, is carried into the canal.

If eddies can thus be prevented by combining a stream-line entry design to the sills of the diversion gates, the horizontal partition may be shortened in length to its remote up-stream edge. It has been stated that the under-sluice passages would choke with submerged driftwood. Analyzing this possibility, the Boulder and the Parker Dams will trap such wood, and any occasional pieces brought into the approaching stream by intervening tributaries with cloudburst run-off, could be found and removed by the regular inspection and duties of the gate operators. Access to the passageways must be provided in the design; a tentative idea is suggested in Fig. 10.

Mr. Collings has estimated that 4 700 sec-ft of sluicing water would be the flow through the suggested under-sluices, and believes that 3 000 sec-ft

[56] *Minutes of Proceedings*, Inst. C. E., Vol. 23, 1931-32, pp. 7-8.

would be adequate if the multiple basins were adopted. The flow through the under-sluices should be controlled by outlet gates, and would be variable depending on the traveling bed load and on the seasonal demand for the diverted water.

For one or more decades after the Boulder Reservoir is first put in operation, a major part of the annual flow of the river will pass the diversion

Fig. 10

structure. The Yuma and California Projects will use about 5 000 000 acre-ft annually, while the yearly run-off of the river varies between 9 000 000 and 25 000 000 acre-ft. Therefore, the question as to whether a few thousand second-feet of surplus water can be made available for the requirements of sluicing flow need not be raised for many years; nor will the market for the potential hydro-electric power be developed toward its ultimate capacity for a long period, to require conservation of all available water.

The writer purposely refrained from venturing into the field of probable effects that may be produced in the lowest reaches of the river by the unwanted sludge discharged through the sluices. Wishing to concentrate attention to the problem of effecting its separation from the diverted canal water is the reason for not making the matter of sludge disposal a feature of the paper. However, he is glad to see the point raised in the discussion.

In the river bed immediately down stream from the diversion structure, deposition of the surcharge of soil above the capacity soil load of the diminished flow may be expected to produce unstabilized gradients and may cause some expenditure for removing any adverse condition against the down-stream toe of the Imperial Dam, resulting from this deposition.

If the sluicing water and by-passed flow for other users of established water rights, is limited to minimum requirements, the usual stream to be expected would be reduced from its former river magnitudes that were laden with silts and sands originating in and above the canyon section of the watershed, to the comparatively small volumes of flow sufficient for a large-sized canal, and carrying a capacity soil load which originates in the 250-mile length below Boulder Dam. The aggregate available area contributing to this soil load, has been stated as about 25 000 acres, from which on an assumed 10-year period for removal, an averaged annual volume of 31 250 acre-ft is estimated; such a quantity is relatively insignificant when compared with the present vast soil volumes annually lodged on the lower delta. Therefore, there should be little apprehension there, as to the future soil quantities that

may be expected, during the life of the Boulder Reservoir; and if the flow is occasionally increased to quantities above these minimum requirements, the ultimate soil volumes to be removed are not changed, and the sluicing soils may be insufficient to contribute capacity soil loads, whence varying degrees of turbidity and bed load are quite as likely in the lower river regions in the future with the lesser stream flows.

The failure in 1931 of a deep steel sheet-piling sea-wall, retaining a fill dredged from the estuary of the Los Angeles River produced investigations in which it was essential to know the correct unit weight of the fill. A mechanical analysis showed fine silts to predominate, 58% passing a No. 200 mesh sieve. The weight of the soil was 120 lb per cu ft, with its water content, as removed from the fill. Engineers and textbooks expressed variant viewpoints as to the unit to be used in calculating the fill pressure. Some assumed that the water contained in the fill (the joints of the sheet-piling not being water-tight) counteracted the ocean hydrostatic pressure of 65 lb per cu ft, using the difference of 56 lb per cu ft as the effective unit loading produced by the fill. This would be an acceptable assumption provided the soil particles were spherical or granular, when the interstices or voids would, of course, be interconnected; but with the lamina-shaped particles coming to rest in still water with their flat sides horizontal, the structuralization of the resulting soil-fill entraps the water in the voids which probably can not be interconnected, such water being an integral part of the fill, and the laboratory weight of 120 lb is that which should be used. Thus, the safety of important structures is sometimes dependent on the weighing of all the physical characteristics of the soil particles, the usual considerations of mechanical analysis and the percentage of voids being insufficient if taken without thought of the structuralization of the particles.

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DISCUSSIONS

WIND STRESS ANALYSIS SIMPLIFIED

Discussion

BY JOHN B. WILBUR, ASSOC. M. AM. SOC. C. E.

JOHN B. WILBUR,[32] ASSOC. M. AM. SOC. C. E. (by letter).[32a]—Step (4) of the simplified method of wind stress analysis presented by Professor Grinter, consists of computing the total shear in each story from the balanced

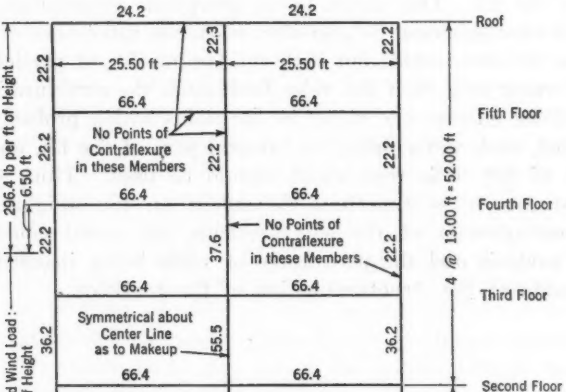

FIG. 17.—DIMENSIONS AND STIFFNESS FACTOR K, OF A BUILDING FRAME (AVERAGE VALUES OF K ARE GIVEN FOR GIRDERS).

moments of Step (3). This is easily done by dividing the sum of the end moments of the columns of a story by the story height, providing one

NOTE.—The paper by L. E. Grinter, Assoc. M. Am. Soc. C. E., was presented at the meeting of the Structural Division, New York, N. Y., January 19, 1933, and published in January, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1933, by Messrs. Francis P. Witmer, Elmer K. Timby, N. A. Richards, John B. Letherbury, Frederick Martin Weiss, and Raymond C. Reese; and May, 1933, by Messrs. A. Floris, Robins Fleming, J. D. Gedo, H. V. Spurr, and Johannes Skytte.

[32] Instructor in Civ. Eng., Mass. Inst. Tech., Cambridge, Mass.

[32a] Received by the Secretary June 3, 1933.

assumes that the wind loads are concentrated at the panel points of a bent. In some cases, however, it might be more correct to assume that the wind load is distributed over the windward column stack of the bent, or partly distributed in this way and partly applied in panel loads.

That the assumption made with regard to this condition may play an important rôle in determining the computed wind stresses in a bent is illustrated by the following: Consider the moments in the columns and girders of the bent shown in Fig. 17 due to the given distributed wind loads. The moments in Table 10(*a*) were determined by means of a model and

TABLE 10.—EFFECT OF VARIATIONS IN THE MANNER OF DISTRIBUTING WIND LOADS

Floor	Wind load	Moments, in Foot-Pounds, Act Clockwise at End of Member if Shown Positive									
		Columns (See Fig. 17)						Girders (See Fig. 17)			
		A		*B*		*C*		*AB*		*CD*	
		Top	Bottom	Top	Bottom	Top	Bottom	Left	Right	Left	Right
(*a*) Moments by Model Test and Deformation Apparatus (Distributed Loads)											
Roof	296.4*	−4 140	−6 810	+200	−9 500	+2 560	−2 185	+4 140	−1 410	+1 200	+2 560
5	296.4*	−15 180	−8 210	0	−31 450	−10 900	−9 460	+21 990	−3 100	+12 170	+13 085
4	296.4* 222.3*	−25 382	−22 128	+17 160	−68 900	−12 890	−11 678	+33 593	+1 877	+12 170	+22 530
3	222.3*	−26 540	−24 260	−41 230	−35 270	+4 800	−40 800	+48 668	+37 730	+72 700	+6 880
2	222.3*	−14 550	−33 820	−59 350	−6 450	−33 700	−68 500	+38 810	+58 200	+34 300	+60 700
Main	222.3*	−18 135	−45 450	−42 600	−67 900	−24 000	−39 010	+51 955	+45 000	+4 050	+92 500
Basement								.....	.....	.....	.....
(*b*) Moments by the Slope-Deflection Method (Equivalent Concentrated Loads)											
Roof	1 910†	−3 600	−2 800	−6 000	−5 900	−3 600	−2 800	+3 600	+3 000	+3 000	+3 600
5	3 830†	−11 900	−10 600	−15 600	−14 100	−11 900	−10 600	+14 700	+10 800	+10 800	+14 700
4	3 720†	−15 900	−14 800	−32 000	−29 600	−15 900	−14 800	+26 500	+23 100	+23 100	+26 500
3	3 010†	−22 200	−20 200	−40 000	−37 100	−22 200	−20 200	+37 000	+34 800	+34 800	+37 000
2	3 010†	−27 700	−27 300	−53 700	−53 000	−27 700	−27 300	+47 900	+45 400	+45 400	+47 900
Main	2 940†	−22 600	−43 100	−41 600	−67 500	−22 600	−43 100	+49 900	+47 300	+47 300	+49 900
Basement								.....	.....	.....	.....

* Units are in pounds per foot of height. † Units are in pounds.

deformation apparatus.[33] These results are unusual in two respects: There is no tendency toward symmetry of stress in a symmetrical bent, and several of the members have no points of contraflexure. These conditions can be explained partly by the fact that the model test takes into account (although incorrectly) deflections due to direct stress and shear. The writer did not believe that the effect of these items was sufficiently great to offer complete explanation for the unusual results. A solution was made therefore by slope

[33] "An Example of the Mechanical Solution of a Statically Indeterminate Structure," by H. W. Coultas and V. H. Lawton, *The Structural Engineer*, October, 1930, p. 344.

deflection, concentrating the same total wind loads at the panel points, and, to simplify the tedious solution, using average moments of inertia for girders with cover-plates. The results of this solution, which was carried out on a computing machine with considerable accuracy, are given in Table 10(*b*), and show such completely different moment values, that the importance of the manner of loading assumed is apparent at once.

In a given building frame, the paths which the wind loads follow in their transfer from exterior walls to wind bents depend on the type of construction used and the ratio of column spacing to story height. For skeleton steel buildings with curtain walls, the assumption that the wind loads are concentrated at the panel points of the bents seems rational to the writer, particularly if the building is tall, or if the ratio of column spacing to story height is large. The possible importance of this assumption in its effects on the computed stresses, however, should not be overlooked.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

METEOROLOGICAL DATA
PROGRESS REPORT OF SPECIAL COMMITTEE

Discussion

BY MESSRS. JOSEPH JACOBS, AND J. A. FRAPS

JOSEPH JACOBS[34], M. AM. SOC. C. E. (by letter)[34a].—The initial progress report of the Special Committee on Meteorological Data is of unusual interest and value, and the Committee merits highest commendation for its excellent assembly and presentation of basic facts concerning the operations of the U. S. Weather Bureau Service and for its frank discussion of deficiencies and possible improvements of that Service. That the successive reports of this Committee (for there is indication that there will be several), together with the discussions that they will arouse, will, inevitably, have a far-reaching and beneficial effect on this very important Federal service, there can be little doubt. The more important immediate considerations are not those which concern the detailed scientific and technical phases of meteorology, nor are they the best methods of observing meteorological phenomena, important as these considerations are, but, rather, they relate to questions of possible changes and betterments in the administration of the Service that will tend to expand its scope and usefulness and make it more immediately and more extensively responsive to the needs of the time.

The purpose of this brief comment, therefore, is not to enter into any discussion of possible controversial questions of scientific meteorology, but simply to express appreciation of the work the Committee is doing and to aid, in a measure, in impressing upon the U. S. Weather Bureau Service the fact that its output is extensively needed and utilized—and increasingly so—by engineers throughout the country and also, necessarily, that it is being constantly and carefully studied and tested by these engineers. It is evident from the replies to the Committee's questionnaire that the Engineering Profession is not satisfied with some phases of the Federal meteorological

NOTE.—This Progress Report of the Special Committee on Meteorological Data was presented at the Annual Meeting, New York, N. Y., January 18, 1933, and published in January, 1933, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: April, 1933, by Messrs, C. S. Jarvis, C. F. Marvin, and Ivan E. Houk; May, 1933, by C. E. Grunsky, Past-President, Am. Soc. C. E.; and August, 1933, by Messrs. Charles W. Sherman, J. C. Stevens, Thorndike Saville, A. L. Sonderegger, and Edward Hyatt.

[34] Cons. Engr. (Jacobs & Ober), Seattle, Wash.

[34a] Received by the Secretary July 17, 1933.

service, and it is just such constructive criticism as is embodied in the report that will aid in hastening the desired expansion and improvement of that service. The writer is confident that the Weather Bureau will welcome constructive criticism and suggestions for the betterment of its service.

How extensive is the utilization, by engineers, of the meteorological data compiled by the Weather Bureau is indicated in Section IV of the report, a dozen specific uses being there listed. To engineers in private practice, particularly in the West where close specialization is less prevalent than in the East, the data for several of those uses are constantly in demand. At one time or another, the writer has found need for all of them, but more particularly for those relating to temperature, precipitation, and evaporation. He has always found the officials of the Weather Bureau promptly helpful in providing all data requested, that were available and that they were able to supply. The chief difficulty encountered has been the insufficiency of observation stations at critical locations as, for instance, in high mountain drainage areas. This is true both for precipitation and for evaporation of which latter there is a special dearth of adequate detailed information in the State of Washington. A better distribution of stations and a more frequent inspection of those stations is needed in this State. It is appreciated that it is practically impossible to find volunteer observers for high mountain stations that are under a heavy mantle of snow throughout the winter, and that funds are apparently not available for paid observers; but these conditions do not alter the fact that such stations are needed and that, sooner or later, they must be established.

Thus far, the major activities of the Weather Bureau have been ordered in respect of the immediate needs of commerce, and serving the needs of engineering has been rather a secondary consideration. The latter, however, is of such expanding importance and its need for refined meteorological data is so great, that it should no longer be viewed as a subordinate branch of Weather Bureau work, but rather as being co-ordinate with its other major activities. Quoting largely from his replies to the Committee's questionnaire, the writer submits the following suggestions as to how the Weather Bureau Service might be made of more value to the engineer:

(*a*) For Western stations, the publication of precipitation data for seasonal as well as for calendar months—at least, the annual totals for seasonal years.

(*b*) More frequent and more detailed statements of maximum storms and rainfall intensities.

(*c*) More frequent inspection of stations, which means not only more thorough and detailed instruction of observers, but also a more prompt correction of instrument defects, station exposure, and observation methods. In the State of Washington stations are inspected at about three-year intervals; they should be inspected at least once a year.

(*d*) The substitution, generally, of ground stations for high-building, roof stations; or, at least, the addition of the former at selected stations,

so that dependable comparative data for ground and elevated stations would be available.

(*e*) The establishment of numerous stations in high-mountain drainage areas, to be observed the entire year.

(*f*) More extensive observations for evaporation, from both ground and water surfaces, for varying exposures, and for varying wind and temperature conditions. There are altogether too few evaporation stations in the Pacific Northwest.

(*g*) More extensive and more detailed observations of transpiration and of interception of precipitation.

(*h*) More extensive snow surveys, which are of high practical value in connection with storage considerations and in forecasting probable run-off for power and general water supply requirements.

It has seemed to the writer that the Weather Bureau has not sufficiently utilized the co-operative possibilities of such agencies as the Postal Service with its numerous offices in remote locations, the Forest Service with its many ranger stations at high altitudes, the railroads and city governments, etc. If college courses would provide for some specialization in meteorological work it would aid the Weather Bureau in demanding higher standards of service and better technically trained men. There are few, if any, colleges offering such special courses at the present time and the writer wonders whether the Weather Bureau is actively trying to correct this condition.

Finally, it should be stated that to increase appropriations is a prime necessity, and it is probable that by outlining the broader service here considered this Committee can be instrumental in securing such increased appropriations. This broader service would involve a material increase in the number of observation stations, more extensive evaporation and transpiration studies, an enlarged research staff operating independently of the regular staff devoted to routine observational and forecasting work, and a more extensive distribution of meteorological data.

J. A. FRAPS,[35] Assoc. M. Am. Soc. C. E. (by letter)[35a].—Experience has shown that any group affected by a governmental activity can be of inestimable value to those responsible for the particular activity by offering constructive criticism of the work. This report has presented a discussion on the activities of the United States Weather Bureau that can be of material assistance in the improvement of that Government service. No one would expect the Weather Bureau to adopt, without further ado, the entire program recommended by the Committee. Many of the Committee's recommendations, however, will gradually come into being, either by direct action on the part of the Bureau, or through the natural development of its work.

Engineers find considerable use for the records published by the Weather Bureau, and generally speak highly of the manner in which records are collected and distributed. The service rendered has been of real value in many studies, and credit has been given when due. It is not reasonable to expect,

[35] Phoenix, Ariz.

[35a] Received by the Secretary July 25, 1933.

however, that engineers have found everything they desired for their studies. In other words, the Weather Bureau can not be classed as a perfect organization, since any of the varied works of man are subject to improvement from time to time. Should any organization reach the point where it felt no further improvement were possible, then a general re-organization would be considered in order.

Nearly all leading industries find that they can afford large sums for research. In the past some producers have prided themselves in thinking that their products could not be improved, but when their competitors came in and took away a large part of their business, they began to see that research and improvements were quite essential.

The finding of the Committee that the Weather Bureau does not indulge in as much research as would be desirable, is well founded. The work incident to collecting and distributing climatological data has not been developed to the point where no further improvements can be made. Even the form of the Weather Bureau report might well be revised from time to time, to conform with changing demands.

The purposes of governmental activities are to promote the peace, happiness, and welfare of the people. If an agency of the Government does not perform these functions, it might well be considered as unnecessary. No one questions the desirability of the work performed by the Weather Bureau, because its aid to agriculture, commerce, and navigation is indispensable. Neither can one argue that the task is the same in all communities. This is especially true in the arid West. Certain data that are essential in the populated industrial centers are of little or no value in a community that is entirely dependent upon irrigation for its prosperity and well-being. It matters little whether 2 or 3 in. of rain falls at many different points in an irrigated valley during the course of a year, but it is a matter of vital importance to know the average annual rainfall on the drainage area that supplies water for irrigation in the valley. A smaller number of stations in more desirable locations would be considered a decided improvement in the service of the Weather Bureau, as far as many parts of the arid West are concerned.

The suggestion has been made that the Weather Bureau should attempt to obtain information which will be of particular value to the area in which records are taken. This, of course, would be in addition to the records needed for the general information of the public, and for the needs of the Weather Bureau itself. As a specific example, records of the intensities and amounts of precipitation on water-sheds in the West would be of great value—greater, in fact, than numerous records in the desert country. These data would assist materially in the determination of the maximum annual yield of water from the area in question, and also the greatest peak discharge from the drainage basin. This would provide records needed by the engineer in designing flood-control structures, or in fixing the acreage that could be irrigated from any specific drainage area. That these two items are important may be seen from the fact that the prosperity of many Western communities

depends very largely on the water supply available, and no community can prosper long if it is subject to frequent and disastrous floods, or to insufficient water for the irrigable acreage.

Evaporation from the surfaces of reservoirs is of no little consequence in the West. A study[36] by Carl Rohwer, Assoc. M. Am. Soc. C. E., led to the conclusion that the type of evaporation pan used by the Weather Bureau was best suited to the study of evaporation from water surfaces, particularly when comparisons are to be made with known data on evaporation losses. Mr. Rohwer also found that the Weather Bureau pan did not give the true evaporation, as the losses from the pans were greater than from a lake or reservoir; and that Weather Bureau evaporation stations did not as a rule obtain sufficient data to permit of direct comparisons with other pans, although approximate data are usually available at some near-by Weather Bureau stations. It would seem that evaporation is of sufficient importance to warrant the compilation of complete data at evaporation stations, and that research should be carried at least far enough to make the Weather Bureau records of the maximum value. Evaporation records are not of great importance if they do not record the evaporation that would occur from a lake or reservoir, or provide fairly definite factors that could be applied to a reservoir or lake near the station.

An expenditure involving millions of dollars annually goes to various forms of flood-control structures scattered throughout the nation. A vast amount of valuable data is furnished by the Weather Bureau to engineers for their studies relative to flood-control works. A greater effort in this connection would be desirable, however, because the matter of controlling disastrous floods is still far from an ideal solution. Rainfall intensities would be of particular value in this connection, and should be obtained whenever possible. If particular care is taken to distribute meteorological stations over drainage basins, the resulting records will be of greater value than at present.

A number of stations have been found to be situated on the tops of high buildings in large cities. This practice evidently results in a difference between actual and recorded measurements, because various observers have found wide, although not consistent, differences. In so far as possible, it would be considered superior to have all stations at ground elevations, in order that there should be no doubt as to the value of the record. In these days of rapid transportation, it does not appear necessary to place Weather Bureau stations in congested areas. If conditions demand that a station be situated at a high elevation above ground, a note to this effect would be considered desirable in all published records.

With centralized authority in Washington, D. C., the Weather Bureau is at a distinct disadvantage when it comes to satisfying local needs. This matter of taking into consideration local conditions should be in addition to furnishing data that are to be correlated for the entire country and for the general information of the Bureau.

[36] *Technical Bulletin No. 271*, U. S. Dept. of Agriculture, p. 88; *Proceedings*, Am. Soc. C. E., February, 1933, p. 229.

Many large industries take college trained technical men, give them a specialized course of study, and place them in responsible positions throughout the organizations. If a procedure of this nature is of value to industry, where the majority of the positions are not purely technical, then a technical organization like the Weather Bureau should find it even more profitable. An organization composed of well-trained technical men would not find it difficult to determine the need for records that would be of particular value to the community in which they worked. It should be the duty of local and State observers to seek the particular needs of the areas in which they work, and to recommend special records for the stations under their control, all subject, of course, to the approval of the Director of the Bureau in Washington.

A further objection to the Weather Bureau relates to its personnel, the Committee finding a dearth of college graduates engaged in the work. This in itself would not necessarily warrant adverse comment, if men in the Bureau were particularly well trained or experienced in their respective fields. Generally speaking, however, the individual who secures a broad technical education through self study is very rare indeed. Too often the training consists of a smattering of general knowledge and varying degrees of proficiency in the special occupation that engages the individual's working hours. There is usually a serious deficiency in background, which restricts the thought and general efficiency of the non-college trained men.

There has been no effort in this discussion to exaggerate the value of Weather Bureau records to engineers. A distinction should be made between the needs of engineers, of local chambers of commerce, of meteorologists, and others for such data. A large part of the engineer's time is devoted to works that are dependent in some measure on Weather Bureau records. Taken as an aggregate these works would reach an astounding total. If the Weather Bureau can improve its service to engineers, it can perform a direct service to the people of the country at large, because every one is affected by structures that are feasible or safe only as long as reliable meteorological data are available for their planning. There is no mercenary motive for a request by engineers for improved Weather Bureau service, but only a desire to perform more efficiently the engineer's service to the public.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

HIGH DAMS ON PERVIOUS GLACIAL DRIFT

Discussion

BY MESSRS. M. M. O'SHAUGHNESSY, AND H. DE B. PARSONS

M. M. O'SHAUGHNESSY,[8] M. AM. SOC. C. E. (by letter)[8a]—This paper is very illuminating with regard to problems in dam construction in the Michigan glacial drift formation. The most interesting part is the introduction of a concrete core into the hydraulic fill, with precautions taken to make the core tight and impervious. Further information concerning the details of the design and construction of this core-wall would be interesting. Where water does not have very great value, a leakage of 3.7 sec-ft is not excessive, provided it does not impair the stability of the dam structure if a substantial volume of water is retained behind the dam.

In Western communities this leakage would be regarded as excessive. Its existence might impair the safety of the structure. For example, in the Priest Dam of the Hetch Hetchy Project, 1 160 ft long and 147½ ft high, the leakage amounts to only 219 600 gal in 24 hr, equivalent to 0.34 sec-ft, and has remained practically constant at that amount for the eight years, since 1925, when the dam was completed. The core-wall was constructed absolutely vertical by having the fills on each side kept at the same height.

The writer does not concur with Mr. Burd's contention that in constructing the Hardy Dam the use of floating equipment, giving access to fines, was undesirable (see heading, "Design and Settlement of Embankments"). The hydraulic method of handling available materials is certainly the most desirable from a cost point of view.

The drainage system on the down-stream base of the Hardy Dam is an excellent one. Its installation was a very admirable precaution. The paper as a whole opens up an interesting field in the economics of dam construction. The writer concurs in all the "Conclusions," except Conclusion (3). The settlement of the up-stream fill of Priest Dam of 2½ ft, and of the down-stream fill of 6 ft, confutes this statement. The construction of Priest Dam,

NOTE.—The paper by Edward M. Burd, M. Am. Soc. C. E., was published in April, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: May, 1933, by Messrs. Charles W. Sherman, and F. B. Marsh.

[8] Cons. Engr., San Francisco, Calif.

[8a] Received by the Secretary, May 3, 1933.

145 ft high, makes Conclusion (6) tenable. Excellent care devoted to the construction of the foundations of the core-wall is very essential to the completion of a satisfactory dam.

H. DE B. PARSONS,[9] M. AM. SOC. C. E. (by letter).[9a]—This is an interesting paper on the construction of dams on pervious foundations, and it contains information which should be useful to engineers who may have to erect structures under similar conditions. In the beginning of the paper, Mr. Burd speaks of the unusually uniform flow of the rivers in the glacial drift in Michigan and draws the conclusion that the hydro-electric field is reduced to supplying peak power. The conditions described are also favorable for the hydraulic plants to supply prime power and for letting the steam plants take the peak loads.

Mr. Burd calls attention to layers of glacial silt and rock flour compacted into solid masses, which he terms "mudstone." Similar material was found in the Hudson River when excavating the foundations of the Spier Falls Dam.[10] The rock dipped sharply on both banks. This V-shaped pocket, about 400 ft wide at the river surface, was approximately 57 ft in depth. The upper portion was filled with coarse sand and boulders, then fine sand, and, at the bottom, very fine dark silt and rock flour, compacted into a solid mass that required blasting. This material was practically impervious, but crumbled into fine dust on exposure to light and air. Its supporting power was very high in its natural state, while buried and held by the geologic rock floor. Practically no boulders were found in this mass. It was considered treacherous and was taken out in order that the base masonry of the dam could rest on the rock.

Mr. Burd refers to the settlement of the foundations in Michigan as being proportional to the load, but the writer's experience has been that true proportional settlement cannot be predicted. Mr. Burd also speaks of fine sand grading down to quicksand, but the writer does not like the word, "quicksand," as there are fine sands which are not "quick." The material was apparently fine sand with "quick behavior," and is always treacherous. The writer acted as Consulting Engineer for the construction of a power house that had to be founded on a bed of sand beneath which was a layer of sand with quick behavior. The work was accomplished by confining the material by steel interlocking sheet-piles, so as to divide the area into rectangles about 41 by 35 ft. These rectangles were excavated alternately until the sand blanket was reached, but the lower layer of sand with quick behavior (which had an artesian pressure) was left undisturbed. Occasional boils formed which were plugged by pouring broken stone (concrete coarse aggregate) into them. The broken stone acted as a reverse filter and soon clogged itself with the finer material brought up by the artesian flow. Such foundations are expensive, but at times cannot be avoided.

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[9] Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

[9a] Received by the Secretary May 19, 1933.

[10] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1326, Fig. 31.

Mr. Burd refers to paving the up-stream face of sand dams with concrete. Concrete facing has not always proved water-tight. The writer had an experience with a canal which was so lined, but when the canal was rapidly dewatered, the hydrostatic pressure on the outside burst the canal lining inward. The leakage, through the concrete lining, remained in the natural sand beds through which the canal was dug. In other words, the drainage through the sand was not rapid enough to remove the head due to leakage. The writer also understands that some rock-fill dams, faced with concrete, have not remained tight, probably due to settlement and shrinkage under their water loads.

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DISCUSSIONS

ACTUAL DEFLECTIONS AND TEMPERATURES IN A TRIAL LOAD ARCH DAM

Discussion

BY GEORGE JACOB DAVIS, JR., M. AM. SOC. C. E.

GEORGE JACOB DAVIS, JR.,[4] M. AM. SOC. C. E. (by letter).[4a]—The author's Fig. 22 and the accompanying data seem to indicate that at Elevation 90 and Elevation 120 the actual deflections of the Ariel Dam agreed with the computed deflections reasonably well when the water was near its maximum elevation and the dam and the lake water temperatures were near their minimum values. At Elevation 180 and Elevation 220 the actual deflections, under the same conditions were, roughly, three-fourths of the computed values at the crown and somewhat more than the computed values at the haunches of the arches.

At the lower elevations only a minor part of the water load is carried by arch action and the deflections, therefore, are governed largely by resistance to shear on a horizontal plane. At the higher elevations the deflection is due largely to bending.

It appears to the writer that the discrepancy between the actual and the computed deflections at the higher elevations may be accounted for by assuming that the modulus of elasticity of the concrete was actually much higher than the value of $E_c = 2\,000\,000$ lb per sq in., which was used in the design.

Figs. 8 and 11 show that at the time the maximum deflections were measured (May 3 to 6, 1932), the compressive strength of the concrete approximated 4 000 lb per sq in., and Fig. 10 indicates that in the interior of the mass of concrete the compressive strength may have averaged 5 000 lb per sq in., or more.

Stanton Walker, Assoc. M. Am. Soc. C. E., has shown[5] that the relation between the tangent modulus of elasticity, at 25% of the compressive

NOTE.—This paper by A. T. Larned and W. S. Merrill, Members, Am. Soc. C. E., was published in May, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[4] Dean, and Prof of Civ. Eng., Univ. of Alabama, Tuscaloosa, Ala.

[4a] Received by the Secretary July 25, 1933.

[5] "Modulus of Elasticity of Concrete", *Bulletin 5*, Structural Materials Research Laboratory, Lewis, Inst., Chicago, Ill.

strength, and the compressive strength of concrete may be expressed by the equation,

$$E_{ts} = 66\ 000\ S^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

from which it may be surmised that if the strength of the concrete in the Ariel Dam was 4 000 to 5 000 lb per sq in., the modulus of elasticity was probably as high as 4 000 000 to 5 000 000 lb per sq in.

It is probable that the Ariel Dam, and all others thus far designed by the trial-load method, are considerably stiffer and stronger structures than they were assumed to be by their designers. Stiffer cantilevers would crack to a less depth, thereby changing the moment of inertia of the cantilever sections and decreasing the hydraulic uplift in the cracks, and stiffer arches would not crack so deeply at the crown and near the abutments. The distribution of the water load between the cantilever sections and the arch sections is, therefore, doubtless somewhat different from that assumed in the design.

Knowledge of the true value of the modulus of elasticity of the masonry is secondary in importance only to knowledge of the true value of its strength, if a rational design is to be made of an arch dam by the trial-load method. The effect of the moduli of elasticity of the materials used in the Stevenson Creek Dam and its foundations was studied by the Committee of Engineering Foundation on Arch Dam Investigation.[6]

It would seem that a careful study should be made also of the effect of the concrete in the core of the dam having a higher modulus of elasticity than that near the faces, for since the interior concrete had a higher strength value it probably also had a higher modulus of elasticity than the surface concrete. Such a difference in the compressibility of the concrete would affect the depth of cracking at the crown and abutments and, consequently, the moment of inertia of the section and the position of the line of thrust.

There may be some advantage in making the blocks of concrete shorter even than the 30 ft adopted in the construction of the Ariel Dam and also in proportioning the slots between the blocks so as to secure the benefits advocated[7] by A. V. Karpov, M. Am. Soc., C. E. It is also possible so to design the slots and to fill them in such a way as to compensate to a considerable degree for the shrinkage of the concrete in the blocks.

An attempt to attain this end was made, in 1928, by the writer in designing an arch dam, for the water supply of Tuscaloosa, on Little Yellow Creek, in Alabama. The blocks of the dam were made 50 ft long each side of a 4-ft "closing plug" which was not placed until the completion of the dam. At the ends of the two 50-ft sections, openings of the shape shown in Fig. 24 were left in the dam. These openings were provided with metal water-stops at the up-stream and down-stream faces of the dam. The openings were made as small as possible, so as to permit of their being completely filled with concrete in one continuous operation requiring only a few hours. They were large enough to permit a workman and an inspector to see that the concrete was properly placed through a tremie and spaded to avoid

[6] *Proceedings*, Am. Soc. C. E., May, 1928, Part 3, p. 153.
[7] *Loc. cit.*, April, 1932, p. 507.

honeycombing. By this means the entire depth of concrete was placed before that at the bottom of the slot, or "expander joint", had set.

The hydrostatic head thus produced in the fluid concrete exerted sufficient pressure on the concrete of the previously cast blocks to compensate for fully one-half the temperature range expected at the base of the dam, and con-

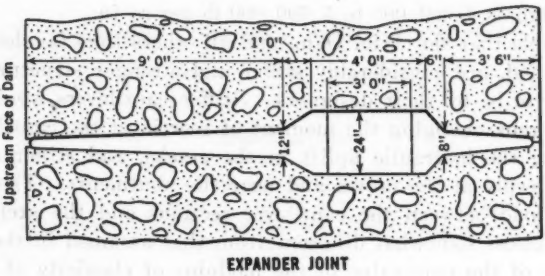

FIG. 24

siderably more than the full temperature range in the upper, more flexible, part of the dam. The expander joints were concreted when the temperature was approximately at its minimum value. Temperatures were determined at three levels in the dam and at three points at each level, by means of thermocouples placed one at the middle and one each 1 ft. from the up-stream and down-stream faces of the dam, respectively. This dam was built elliptical in plan with increasing axes at the higher levels. The longer axis was parallel to the axis of the valley. In this way and by means of the expander joints an attempt was made to relieve the dam of part of the stresses due to contraction of the concrete, an idea which has since been developed in a more definite way by Mr. Karpov.

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DISCUSSIONS

EARTHS AND FOUNDATIONS

PROGRESS REPORT OF SPECIAL COMMITTEE

Discussion

BY MESSRS. DANIEL E. MORAN, AND A. E. CUMMINGS

DANIEL E. MORAN,[15] M. AM. SOC. C. E. (by letter).[15a]—This report contains so much of interest and value for all engineers interested in foundation design that it seems almost ungracious to criticize it. Gratifying progress has been made along several lines, and the investigations and tests presented in the report are of great value. Nevertheless, it can scarcely be claimed (as stated in the report) that the application of this knowledge to specific problems by engineers has been made generally practicable. The problems connected with foundation design are difficult for many reasons, such as the great diversity in the materials encountered and their radically different physical qualities, and because there is difficulty in predicting the behavior of these materials when stressed; and furthermore, because the analysis must be volumetric or three-dimensional rather than linear.

Through the devoted efforts of a number of investigators in this field, among whom the members of this Committee are especially prominent, the knowledge of the physical characteristics and behavior of soils under stress has been advanced rapidly in the past few years. Valuable as these results are in guiding engineers in their judgment, the time has not yet arrived when it can be said that definite calculations can be made, justifying the prediction of the exact settlement of a given structure on a given site.

The engineering problems presented, in common with all other engineering problems, involve material; that is, matter as well as force, motion, and time. The material involved may be any one of a number of natural deposits found in the crust of the earth, which in a given site may be counted on to differ in some degree from the materials in any other site. Moreover, in a single site, there may be great variations in the character of the material

NOTE.—The Progress Report of the Special Committee on Earths and Foundations was presented at the Annual Meeting, New York, N. Y., January 18, 1933, and published in May, 1933, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: August, 1933, by Messrs. L. C. Wilcoxen, H. de B. Parsons, William P. Kimball, and T. A. Middlebrooks.

[15] Cons. Engr. (Moran & Proctor), New York, N. Y.

[15a] Received by the Secretary, August 5, 1933.

found at different depths and in different areas. These variations in the natural materials greatly increase the difficulty of design, of predicting settlements accurately and, finally, of connecting the actual settlements observed with the causes therefor. In no other branch of construction is the engineer called on to use materials the character and qualities of which he knows so little.

The report very properly does not attempt to furnish engineers with a textbook on foundations and should be considered as a general review of recent developments under different headings and classifications. The writer will so consider it in the following discussion.

Under the heading, "The Basic Problem," an elemental example is presented for preliminary study, in which calculations are based on piles of rollers and balls. This elemental example is evidently discordant with the known facts, and no consideration is given to the lateral pressure exerted between the rollers or balls. The pressure distribution shown in Fig. 1 is discordant with the results of tests and with the result obtained by the use of the Boussinesq formula. The writer does not understand why this example was introduced in the report since apparently no use is made of it, and all calculations as to stresses produced are obtained from the Boussinesq formula.

The Committee passes then to a consideration of pressures due to concentrated loads, basing its analysis of the subject on the well-known Boussinesq formula. A general statement is made that the pressure distribution both in the case of pressures produced by a single applied load and by the pressure of a transmitted load at the surface is independent of the type of material, and that it involves no elastic constants.

The writer confesses freely that he has not mastered the mathematical development of the Boussinesq formula, but understands that it was based on certain definite assumptions as to the character of the material to be subjected to stress. The material considered by Boussinesq was assumed to be elastically isotropic and otherwise uniform in three dimensions and to obey Hooke's law in tension and compression. The bakelite material tested at Columbia University may approximate these assumed characteristics quite closely, but the natural beds of granular material differ widely from Boussinesq's assumptions. A few essential points of difference may be pointed out for beds of sand and clay.

Natural sand beds are composed of separate grains of mineral matter which should be regarded as non-coherent. Therefore, the mass must be considered as incapable of resisting tension in the ordinary sense in which the word, "tension," is used, The grains composing the mass vary in size and shape and, in general, the individual grains show a marked difference in the lengths of their axes. Sand deposits are generally stratified so that adjacent layers differ from each other in size of grain and other characteristics and, moreover, the grains in each layer are generally arranged so that their long axes lie in the same plane. When stressed, the individual grains deform elastically, but the linear deformations resulting from stressing a series of grains does not follow the straight-line law of elastic deformations in solids.

In clay beds the case is somewhat different. Clay beds with which the writer is familiar are distinctly stratified in their natural condition, and have a definite but low tension value. Moreover, the mass is colloidal in character and loses volume when compressed by the slow expulsion of excess water. The stress may be divided temporarily at least between the contained water and the solid matter in the mass.

In view of the radical differences in these natural materials considered in this report from the theoretical material assumed by Boussinesq in his mathematical discussion, there is room for grave doubt as to whether the Boussinesq formula can be safely relied to give the actual stress distribution in natural granular materials.

In addition to the vertical downward component of stress produced by a concentrated load, the Boussinesq formula gives components of stress in a direction radial from the center of the load and in a direction at right angles thereto. In all calculations and in all discussions contained in the report reference is made only to the vertical component of stress in the soil. No where in the report does the possibility of lateral motion of the soil, or indeed, of its lateral compression or consolidation seem to be considered. Settlements of structures may result from motion of the solid and liquid constituents of the soil laterally or laterally and upward. The Committee seems to be entirely silent as to the effect of lateral stress on consolidation or lateral motion.

It may be pointed out that in the Boussinesq formula the vertical component of stress at every point in the foundation bed is downward which is difficult to reconcile with the commonly observed fact that in many cases of settlement, both in wet and dry sand and in clays, material adjacent to the loaded area moves upward. All measurements made of vertical downward components indicate that these components reduce to zero not far from a line drawn from the loaded edges of the area, downward and outward at an angle of 45 degrees. Since the material outside this 45° line is stressed laterally, it follows that the direction of stress at points along the 45° line must be horizontal, and this indicates the probability of upward components of stress outside the 45° line. This view is substantiated further by observations made on the motion of dry sand when subjected to vertical loads such as were observed in experiments made by the writer.[16]

When the soil adjacent to the footing rises above its original level the uplift of soil cannot be the result of downward stress on that part of the foundation bed, and must be due to the effect of lateral stresses producing an uplift. Notwithstanding that the previously quoted considerations and facts are widely known, the Committee neglects entirely the possibility and danger of lateral stress and motion, and provides no assistance to an engineer in guarding against failures from lateral movement.

Under the heading, "The Mechanics of Settlement in Clay," much interesting matter is discussed. The basis of analysis and the methods of calculation are derived largely from the work of Professor Terzaghi, with extensions

[16] "Experimental Loading on Granular Materials," by Daniel E. Moran, M. Am. Soc. C. E., *Journal*, Franklin Inst., No. 1192, April, 1925, Vol. 199, p. 493.

and simplifications by Professor Gilboy and Mr. Buchanan. The fundamentally important "coefficient of consolidation" is measured by a device, a recent form of which designed by Professor Terzaghi is illustrated. A somewhat different apparatus was designed by the writer's office for use in consolidation tests made by the University of California on material from the line of the proposed Trans-Bay Bridge, San Francisco, Calif.[17] In the writer's opinion, it includes some improvements on the device shown in the report.

Consolidation tests should be made, as recommended by the Committee, on undisturbed materials, a practice which has always been followed by the writer. This important point is emphasized by Mr. Casagrande's findings of a radical difference in results of consolidation tests on clay in its natural and in its remoulded state. His experiences have been checked by tests recently made on San Francisco Bay clay and on Mississippi River clay, from New Orleans, La., in the laboratory of the writer's firm by William P. Kimball, Assoc. M. Am. Soc. C. E.[18]

In making consolidation tests it is important to secure an undisturbed sample and, for this reason, the writer believes that it is inadvisable to use the entire sample obtained by a sampling tube on account of deformation of the material adjacent to its periphery caused by forcing the sampling tube into the clay. He also believes that even if the submerged material is water-tight, as in the case of clays, the sample is preferably taken through water maintained at its natural level in the drill pipe rather than through a dry hole.

The Committee, very properly and ably, argues the importance of recording for the general information of engineers the observed settlements due to full-scale loads, and presents a simplified record form for reporting actual cases. In addition to the reluctance on the part of engineers and owners to make public information on the settlement of structures, there is a further difficulty which to his chagrin has frequently prevented the writer from obtaining proper records of the settlement of foundations which were designed and constructed under his supervision. The difficulty referred to is that of maintaining accurate level points on the structure during and after construction. Points established on elements of the footing, such as foundation mats, steel grillages, column bases, etc., are usually covered and lost in the construction of the building. It is generally difficult to arrange to transfer these points to permanently available bench-marks on the finished structure because the engineers interested in the foundation work are frequently not on the work during the subsequent construction. The loss of the original points involves a gap in the record at a most important time as far as settlements are concerned. In many structures difficulty is experienced in arranging for the maintenance of permanent bench-marks and in making arrangements for periodic level readings. However, much may, and should be, done to overcome these practical difficulties.

In conculsion, the writer congratulates and thanks the Committee for its excellent report.

[17] Not yet published. To appear in *Engineering News-Record.*

[18] *Proceedings,* Am. Soc. C. E., August, 1933, p. 1059.

A. E. CUMMINGS,[19] ASSOC. M. AM. SOC. C. E. (by letter)[19a].—This report is an interesting and important contribution to the study of the settlements of buildings and other structures. The problems involved in the study of foundations appear to be similar to some extent to those of the design of other structural members. It is necessary to know the stresses produced by the given system of loading and then to calculate the deflections that will be caused by these stresses. However, the methods of calculating stresses and strains in ordinary structural design are well defined and have been carefully checked by experiment. The materials of construction, to which these design methods are applied, are the results of many years of effort to produce uniformity such that physical properties and actions under stress are known or can be controlled. In the problems of foundation design this is not the case. There appear to be notable differences of opinion as to the distribution of stresses in soils, and there is an almost infinite variety of soils for each of which the physical properties must be determined.

For the determination of the stresses produced by foundation loads, the Committee makes use principally of one of the equations derived by Boussinesq. It should be noted that Boussinesq's work[20] consists of the derivation of certain mathematical expressions for the stresses and strains produced in infinite solids which are subjected to various forms of surface loadings. However, the analysis is purely theoretical and the equations apply only to the elastic isotropic solids. When the equations are applied to foundation studies the question arises as to whether or not soils can be classified as elastic isotropic solids. Most soils have some elastic properties, but the laminated soils are not isotropic. The soils on which structures are usually built generally consist of strata of different materials, such as sands and clays and silts, all of which vary in their elastic properties. Furthermore, the entire mass of soil to which the stress distribution calculations are applied is not isotropic because of its stratification.

The statement is made that stress distributions calculated with this Boussinesq equation have been substantiated by experiments. As far as the writer knows, no experiments have ever been made to determine the distribution of stresses in natural beds of clay. The principal reason for this is probably the practical impossibility of placing pressure capsules in natural clay beds without disturbing the clay. However, many experiments have been made to determine pressure distributions in sand and sand clay mixtures, the most notable of which are probably those made in Germany by Professors Kögler and Scheidig.[21] These experimenters worked principally with sand, using beds of various depths to about 6 ft. The loads were applied by means of small steel plates at the top surface, and the resulting pressures at the bottom were recorded on a pressure-recording device of their own design. As a result of a very considerable amount of experimentation,

[19] Dist. Mgr., Raymond Concrete Pile Co., Chicago, Ill.

[19a] Received by the Secretary August 14, 1933.

[20] "Application des Potentiels," par J. Boussinesq, Paris, 1885.

[21] "Druckverteilung im Baugrund," *Die Bautechnik*, 1927, Heft 29 und 31; 1928, Heft 15 und 17; 1929, Heft 18 und 22.

Professors Kögler and Scheidig determined that the distribution of stress could be pictured approximately as shown in Fig. 53. Directly under the load is a bulb-like contour to which they gave the name "Region of Disturbance." Just below and outside of this is a "Region of Undisturbed Position" and running outward from the edge of the load is a "Boundary Curve" which separates the "Region of Undisturbed Position" from a "Region of No

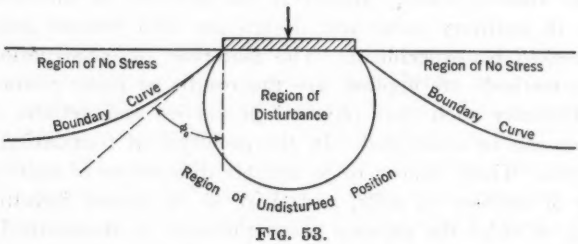

FIG. 53.

Stress" which lies between the "Boundary Curve" and the surface. The "boundary curve" starts away from the edge of the loaded area making an angle with the vertical, which they term, ϕ_o. The value of this angle lies between 40° and 90°, depending on the kind of material to which the load is applied; but the angle is approximately constant for a given material.

Professors Kögler and Scheidig state definitely that in the "region of disturbance" the equation of Boussinesq does not apply. Using the notation of Fig. 2 of the report, the Boussinesq equation for the vertical component of stress may be written in the form:

$$p_z = \frac{3P}{2\pi Z^2} \cos^5 \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

For the vertical stress component within the "region of disturbance," Kögler and Scheidig give the following formula:

$$p_z = \frac{3P}{2\pi Z^2} \frac{(\cos\phi - \cot\phi_o \sin\phi)}{(1 - \cos\phi_o)} \cos^4 \phi \quad \ldots\ldots\ldots\ldots\ldots (41)$$

in which, ϕ_o is the angle made by the "boundary curve" (Fig. 53) and the other letters have the same meanings as they have in Equation (40). The mathematical derivation of Equation (41) is due to Strohschneider[22].

There are several conditions about Equation (41) that should be given consideration. First, there is the fact that it has been checked by considerable experimental work and, therefore, should be considered at least as valid as the theoretical equation of Boussinesq; and, second, there is the fact that it contains several functions of the angle, ϕ_o, and this angle depends on the nature of the material under consideration. It seems reasonable to think that the distribution of stress in a granular material such as sand might differ from that in a material such as clay and that both these stress distributions might differ from that in a material such as rock. The Kögler and

[22] *Sitzungsberichte* der Akadamie der Wissenschaften in Wien, Mathem. Naturw. Klasse, 1912, Abt. II-*a*., p. 299.

Scheidig Equation (41) takes this into account by means of the angle, ϕ_o, but the Boussinesq Equation (40) contains no elastic constants and, therefore, leads to the same picture of stress distribution in all types of material although it should be applied only to the elastic isotropic solids.

A further comparison of Equations (40) and (41) will show several other interesting facts. For elastic isotropic solids, ϕ_o is 90 degrees. The cotangent and cosine of 90° are both zero, and if zero is substituted in Equation (41) for these two functions of ϕ_o. it can be seen that Equation (41) becomes Equation (40). When a calculation is made with equal values of ϕ and ϕ_o in Equation (41) the value of p_z is zero, because the expression in parenthesis in the numerator of the right-hand side of the equation will vanish. Equal values of ϕ and ϕ_o lie on the "boundary curve" (Fig. 53). For values of $\phi > \phi_o$ the value of p_z is negative because cot ϕ_o sin ϕ will be greater than cos ϕ; but points where $\phi > \phi_o$ would lie in the "region of no stress," and Equation (41) is not intended to be used in this region. For small values of ϕ—that is, for points near the vertical center line of the load—Equation (41) gives values of p_z that are greater than those given by Equation (40); how much greater, depends on the value of ϕ_o, which, in turn, depends on the nature of the material. When ϕ_o is about 40°, Equation (41) gives values of p_z that are three or four times as great as those of Equation (40). As ϕ_o increases toward 90°, the values of p_z for points near the vertical center line gradually decrease until at $\phi_o = 90°$, the values of p_z calculated by Equations (40) and (41) are the same, since the two equations are then identical.

This latter fact appears to be important since it indicates that, as far as these experiments are concerned, the Boussinesq equation does not give the correct stress values for points immediately below the loaded area in the "region of disturbance." Kögler and Scheidig do not give any method by means of which the extent of this "region of disturbance" may be calculated. They state simply that it depends on the nature of the soil and on the intensity of the load. In several recent papers on the subject of soil mechanics, the statement has been made that Kögler and Scheidig's experiments show the Boussinesq equation to be valid at depths greater than 6 ft. This conclusion cannot properly be drawn from the work of these investigators. In their experiments, the "region of disturbance" did not extend to a depth greater than 6 ft. because the experiments were performed with small-scale apparatus and small loaded areas. In the case of a large footing, or an entire building foundation, it is probable that the "region of disturbance" extends under the entire building site to a considerable depth. If this is the case it would appear that the ground is subjected to stresses that might be considerably greater than those calculated with the Boussinesq equation.

Many other experiments have been made by other investigators to determine the stresses transmitted through soils. There are the experiments of Spangler, Mason, and Winfrey[23] on the loads transmitted to culverts through highway fills. These experimenters made theoretical calculations with the Boussinesq equation to check the results of their tests. In general,

[23] *Bulletin No. 79*, Iowa Eng. Experiment Station, Ames, Iowa.

their measured stresses were only about one-half as great as the calculated stresses, and their conclusion regarding their static load tests was that the theoretical formula gives the maximum possible percentage of load that would be transmitted through any thickness of fill. This conclusion is just the opposite of that of Kögler and Scheidig whose equation for points below the loaded area gives stresses much greater than those of the Boussinesq equation.

It would seem, therefore, as if there is a reasonable doubt as to the accuracy of the Boussinesq equation for the determination of the transmission of stresses through soils. If the actual stress at a given point can vary from one-half the value to three or four times the value of the calculated stress, there are likely to be large discrepancies between the calculated settlement of a building and its actual settlement. Although the majority of these experiments were made with granular materials, whereas the settlement theory deals with clays, it should be remembered that in many cases the transmission of stress is through granular materials on to clay so that the experiments have considerable bearing on the problem.

For the determination of the amount and rate of settlement the Committee outlines a theory which is generally called the "consolidation theory." The methods of this theory can be applied only to settlements caused by the relatively slow consolidation of clays or very fine-grained soils which are entirely saturated and are submerged below the ground-water level. About 1923 Professor Terzaghi[24] first applied Fourier's diffusion equation to the determination of the rate of consolidation. Several years afterward[25] he developed a somewhat different method for determining the settlement at any given time during the process. Both these methods are based on principles of hydraulic flow and, in order that they may be used, it is necessary that the soil under consideration be completely submerged and saturated.

In many parts of this country and especially in the vast glaciated areas of the Middle West are great beds of clay which do not satisfy the requirements of saturation and submergence. These clay beds are often found high above natural watercourses, such as rivers and lakes, and although there is always some moisture in the clay it is by no means saturated. This being the case it is fairly evident that when loads are applied to these clays there would be no such phenomenon as "hydro-dynamic excess" as described in the report. The action of the clay under load is more nearly like that of an elastic body and, while there is probably some adjustment of the moisture content, this is not the controlling factor in the settlement.

For the determination of the probable settlements of structures built on these relatively dry soils, some method of calculation is required other than that given by the Committee. The problem apparently involves the determination of the elastic constants of the soils and also the establishment of equations for the displacements. The displacements could then be calcu-

[24] *Sitzungsberichte* der Akadamie der Wissenschaften in Wien, Mathem. Naturw. Klasse, 1923, Abt. II-*a*, p. 125.

[25] *Journal of Mathematics and Physics*, Vol. 8, No. 4, 1929.

lated by substituting in these equations the computed stresses and the experimentally determined elastic constants. With this end in view much experimental work has been done, the most notable of which is probably that of John H. Griffith, M. Am. Soc. C. E.[26] Working with clays and loams in their natural states, Professor Griffith made many tests in tension, compression, and shear to determine the strengths of these soils and also their elastic constants. He also made a series of tests to determine the relation of settlements with time, which tests were conducted over a period of about two years. All these tests indicate that these relatively dry soils behave to a considerable extent as elastic bodies, and that the time-settlement curves do not have the characteristic shape of the settlement curves of the consolidation theory. Some experimental work of a similar nature has been done[27] by W. S. Housel, Assoc. M. Am. Soc. C. E. Professor Terzaghi[28] has also conducted a series of experiments to determine the elastic properties of soils. He found that Poisson's ratio and Young's modulus for clay varied with the water content of the clay.

It would seem, therefore, that two distinct theories are required for the determination of the probable settlements of structures. These two theories could utilize the same methods for the calculation of stresses produced in the ground by the given loading. However, after the stresses have been calculated the determination of the settlements of clays and fine-grained soils which are completely saturated requires a consolidation theory in which the settlements depend on the hydraulic flow of water from the soil. For any granular soil, and also for clays and fine-grained soils which are not saturated, it is necessary to use an elastic theory in which the settlements depend on the elastic properties of the soil.

The Committee refers several times to a phenomenon described as the "remoulding effect" produced by the driving of piles into clay. The writer is reasonably certain that to the present time no specific effort has ever been made to prove or disprove the theory of this so-called "remoulding effect." The statements made in the report might lead one to conclude that any building erected on piles driven into clay would be doomed to excessive settlements. Considerable experience with this type of construction indicates that this is not the case. Throughout the great clay beds of the Middle West are hundreds of structures supported by piles of all kinds driven into many different types of clay. While it is true that many of these structures may have settled to some extent, the vast majority of them have experienced such small setlements as to be scarcely detectable, and cases of excessive settlement are the exception rather than the rule.

Although it may not be generally known, there is a phenomenon which occurs during the loading and testing of foundation piles which is a fairly good indication of whether or not the soil has been remoulded. This is the rebound that occurs when the test load is removed from the pile. The writer

[26] *Bulletin No. 101,* Iowa Eng. Experiment Station, Ames, Iowa.

[27] *Engineering News-Record,* February 23, 1933, p. 244.

[28] *Sitzungsberichte* der Akadamie der Wissenschaften in Wien, Mathem. Naturw. Klasse, 1923, Abt. II-*a*, p. 105.

has found many cases in which piles driven into clays have been known to rebound, when the load was removed, to the extent of 100% of the settlement under the test load. This rebound is a common phenomenon especially in heavy glacial clays and whenever it occurs it indicates that remoulding has not taken place to any considerable extent since remoulded clays when tested recover little of their deflections after the load is removed.

In connection with the specific examples of settlements which are given in the report it seems desirable to point out the fact that the theoretical calculations were made after the particular structure was known to have settled. The calculations, therefore, represent an attempt to check an actual settlement, and examples in which the theoretical calculation was made in advance of construction are extremely rare. The assumptions required to reach the results plotted in Fig. 17, present an interesting case. The calculations leading to Table 6 gave results which did not check the actual settlement. It then became necessary to make new assumptions which would check it, and these new assumptions related to the remoulding effect of the piles and the distribution of stresses from the piles to the surrounding soil. Instead of these assumptions, let it be considered that the piles did not remould the clay, but that they had the effect of lowering the plane of the load closer to the top surface of the clay. Let it also be assumed that Kögler and Scheidig's experimental work should be given consideration and that the actual stresses in the ground are somewhat greater than those calculated with the Boussinesq equation. Let it then be considered that the combined effect of these two assumptions is that the loads produced by the building will be just twice as great as those shown in Fig. 15. If the calculation is repeated on the basis of this assumption the total settlement becomes 16 in. instead of 9 in. The ratio, v, becomes 0.85 and J becomes 0.09. The settlement at the end of 1 year is $1\frac{1}{2}$ in.; $9\frac{1}{2}$ years, 6 in.; and 16 years, 8 in. In other words the actual settlements to date can be checked reasonably accurately with an entirely different set of assumptions.

The question then remains as to whether the building will altimately settle $29\frac{1}{2}$ in. or only 16 in. Just how a final settlement of $29\frac{1}{2}$ in. can be arrived at from the data set forth in the report is not quite clear. Fig. 16 (*d*) is presented as a true picture of the stress condition under this foundation, and these stresses may be used instead of those of Fig. 15 in making a settlement calculation. Even if the entire clay bed is assumed to be remoulded, and if the voids ratios are taken from the remoulded curve of Fig. 10, the final settlement calculated with these stresses and voids ratios, by the method outlined in the report, is only about 14 in.

It was in connection with this building that the phenomenon of the remoulding effect of piles was first propounded in order to explain the excessive settlements that were taking place. However, there is as yet no adequate theory for the distribution of stresses around a foundation pile; neither is there any definite information as to the extent to which pile-driving can remould clay. It is not likely that the driving of piles will remould clay to the same extent as the manipulation of small clay samples in a laboratory.

Likewise, in view of the number of buildings which have similar foundations and which are not settling excessively it would seem that the most that can be said about the remoulding effect of piles is, "not proven."

The tank foundation, Case *F*, represents a condition that was studied by Boussinesq,[29] who analyzed the case of a uniform distribution of pressure over the entire interior of a circle on the surface of an elastic isotropic solid. His equations for the deflection represent a concave figure which would be a figure of revolution around the vertical center line of the loaded area. These equations contain two elastic constants and apply to elastic isotropic solids. The action of this tank foundation indicates that the soil on which the tank was built was somewhat lacking in elastic isotropic properties.

The Washington Monument described in Case *G* is another interesting case. A more complete description of this Monument and its foundation has been published by Maj. D. H. Gillette.[30] The boring data indicate that the clay bed under the north edge of the foundation is about 40 ft thick, whereas that under the south edge is only about 20 ft thick. The record shows a total settlement of about 6 in. which is the average of the four corners. If N represents the settlement of the north edge and S, that of the south edge, an equation for the average settlement, in inches, can be written in the form,

$$\frac{N+S}{2} = 6 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (42)$$

According to Equation (16) of the report the settlement is directly proportional to the thickness of the clay stratum. From this, another equation can be written in the form,

$$N = 2\,S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (43)$$

since the clay is twice as thick on the north side as it is on the south side. These two equations solved simultaneously, give $N = 8$ in. and $S = 4$ in.

The application of Equation (16) to this foundation would require a voids-ratio curve for the particular clay that underlies the Washington Monument and such a curve is not available. However, a complete calculation would lead to an even greater differential settlement than is indicated by Equations (42) and (43), because of the differences in the stresses at the top and bottom of the clay under the two edges of the Monument. In other words, the theory requires this Monument to be settling unevenly and to be considerably out of plumb toward the north. The fact is that the Monument is settling quite evenly and is practically plumb.

The aplication of the photo-elastic method to the study of foundation problems is subject to certain limitations several of which are mentioned in the report. The method can be applied only to two-dimensional problems and in some cases, at least, the stress pattern is affected by the supports required at the boundaries of the test specimen. Of more importance is the fact that the photo-elastic method can be applied only to elastic isotropic solids. Figs. 36 and 37 represent a condition that is rarely encountered in foundation

[29] "Application des Potentiels," p. 139.
[30] *Engineering News-Record*, April 20, 1933, p. 501.

construction. These diagrams show stress patterns produced by a bakelite footing on a subsoil of the same material. The physical properties of foundation material usually differ widely from those of the soils on which the foundations are placed, and the soils may be elastic, plastic, or granular. The fact that the pressure at the under side of the footing is non-uniform is fairly well known, and the extension of this non-uniform pressure upward into the footing as well as downward into the subsoil is simply a consequence of Newton's third law.

In Fig. 40(*b*) are shown the trajectories of the principal stresses in the two-dimensional problem, and the direction of the stress arrows on the small element indicates that one of these stresses is to be considered as tensile and the other as compressive. In 1922, George Paaswell, M. Am. Soc. C. E., showed[31] that the trajectories of the principal stresses are families of orthogonal ellipses and hyperbolas with foci that are the extremities of the loaded strip. After discussion, it was concluded that the stresses were both compressive, rather than of opposite sign. In other words, the principal vertical stress is directed downward, and the principal horizontal stress is directed outward, from the vertical center line of the load.

However, it appears that in the three-dimensional problem there is a limiting cone of compression outside of which the direction of the principal horizontal stress is reversed. The equation given by Love[32] for the extent of this cone was taken from Boussinesq[33] who developed the mathematical theory. According to Boussinesq when an infinite solid is subjected to a purely normal surface load there is a cone of compression extending downward into the solid. This is a right circular cone, the apex of which is at the point of application of the load, the axis coinciding with the line of the applied force. The equation by which the extent of this cone can be calculated is,

$$\cos^2 \alpha + \cos \alpha - \frac{\mu}{\lambda + \mu} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (44)$$

in which, α is the half-angle of the cone, and λ and μ are two of the elastic constants of the material under consideration. For Poisson's type of matter in which, $\lambda = \mu$, and Poisson's ratio is $\frac{1}{4}$, the angle α, is about 68° 30′, and the full angle of opening of the cone is about 137 degrees.

The importance of this cone is that it defines a separating surface inside of which the horizontal displacements are directed outward from the vertical axis. Outside the cone the horizontal displacements are directed toward the axis. The Special Committee of the Society on the Bearing Value of Soils for Foundations reported[34] that in the case of an elastic solid the outer region of pressure and tension contributes to the distribution of stress, while in the case of soils, which cannot support tension in any marked degree, the

[31] "Transmission of Pressures Through Solids and Soils," by George Paaswell, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1563.

[32] "The Mathematical Theory of Elasticity," First Edition, 1892, p. 271.

[33] "Application des Potentiels," p. 99.

[34] Progress Report of the Special Committee on the Bearing Value of Soils for Foundations, *Proceedings*, Am. Soc. C. E., August, 1920, Papers and Discussions, p. 933.

surface between these regions presents a surface of discontinuity of strain resulting in a greater concentration of stress within the compression cone. This cone, of course, will have different angles for different kinds of materials, but it is probable that it will not differ widely in ordinary soils. Although this report was written seven or eight years before Kögler and Scheidig published the results of their experiments, it is to be noted that the Committee expected stress concentrations within the compression cone which would exceed the stresses calculated from the theoretical equations, and the experimental work of Kögler and Scheidig demonstrated this to be true. The Committee apparently expected the half-angle of this cone to be approximately constant for ordinary soils, but it is evident from the appearance of two elastic constants in Equation (44) that the angle of the cone depends entirely on the elastic properties of the soil. The picture of stresses and strains presented by this analysis of Boussinesq can also probably account for the "region of no stress" shown by Kögler and Scheidig outside their boundary curve. In an isotropic elastic solid the stress distribution in this region would be due principally to tensile stresses and with a granular material which cannot resist tension no stress would occur.

The reflected bulbs of stress which appear between the loads of Fig. 38 and Fig. 39 may be accounted for to some extent by a two-dimensional phenomenon similar to this three-dimensional cone of compression. The patterns of the photographs represent shears which are proportional to the differences of the principal stresses. The trajectories of the principal stresses intersect each other orthogonally, and those of the shears lie along the bisectors of the angles of intersection of the stresses, as can be seen from Fig. 40(*b*) and Fig. 40(*c*). For a single load the stress pattern is fairly simple, but for multiple loads, especially in the spaces between the loads where the stresses overlap, the stress pattern becomes extremely complicated. It is probable that in the region between the loads and near the surface, the stresses combine so as to produce both tension and compression, and it would seem that some complicated stress pattern of this kind is required to explain the closed ovals in the reflected bulbs of Fig. 38. Just how the stress patterns produced by these multiple loads can be applied to the case of a pile foundation is not quite clear. These loads are purely normal loads applied to the edge of a two-dimensional solid. In the case of a pile foundation the piles are embedded in a three-dimensional solid and transmit their loads to the solid principally by tangential friction forces acting over the entire surface of the pile. The two methods of loading are entirely different. There is also a reasonable doubt as to whether it would be possible to demonstrate the existence of these reflected bulbs in soils. The only experiments made with multiple loads on soils with which the writer is familiar are those[35] of M. L. Enger, M. Am. Soc. C. E., on the stresses produced in railroad ballast by a series of loaded railroad ties. The reflected bulbs did not appear in these experiments, due probably to the fact that no tension could be developed in the granular material with which the experiments were made.

[35] *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1409.

The writer's sole purpose in submitting this discussion is simply to suggest a few words of caution in connection with these various theories. The equations and tables of constants presented in the report might lead an unsuspecting engineer to attempt the analysis of a foundation problem as a more or less straightforward proposition. The problem, however, is by no means as simple as it appears. It is still an open question as to whether or not the stress distribution equations for elastic isotropic solids can be applied quantitatively to soils. It is also to be noted that structures do not always behave in accordance with the mathematical equations of the settlement theories. On the other hand, the entire theory in its present form is far in advance of any other known method of handling foundation problems. It is to be hoped that the Engineering Profession will heed the requests of the Committee and will submit to those interested such information as becomes available from time to time relating to the settlements of structures.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from **September 15, 1933.**

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years of important work
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years of important work
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years of active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

ANDERSON, JAMES DONALD, Pine Bluffs, Wyo. (Age 23.) Oil Inspector, Wyoming State Highway Dept. Refers to R. D. Goodrich, E. K. Nelson, N. T. Olson, H. T. Person.

ANTHONY, THOMAS CRAWSHAW, Jr., Reading, Pa. (Age 21.) Refers to S. C. Hollister, W. E. Howland, G. P. Springer, R. B. Wiley.

ASK, REYNOLD EVERETT, Washington, D. C. (Age 25) Scientific Aide, Bureau of Standards. Refers to O. B. French, J. R. Lapham, V B. Phelan.

ATKIN, MORRIS, Linden, N. J. (Age 21.) Refers to H. N. Lendall, W. Rudolfs, J. Verner.

BALDWIN, WILLARD JUSTUS, Colebrook, N. H. (Age 22.) Refers to E. W. Bowler, G. W. Case.

BALSAM, ARTHUR MICHAEL, San Francisco, Cal (Age 37.) Refers to J. W. Green, J. R. Kelsey, F. E. Leefe, J. C. Stevens, D. R. Warren.

BENNETT, ROBERT NOBLE, McComb, Ohio. (Age 26.) Refers to G. H. Elbin, A. R. Webb.

BIERMANN, ARTHUR EDWARD, St. Louis, Mo. (Age 23.) Refers to E. O. Sweetser, J. L. Van Ornum.

BOLLMAN LESLIE WEBER, Long Island City, N. Y. (Age 24.) Transitman, Stephen J. Reid, Inc., Flushing, N. Y. Refers to E. G. Haines, H. Holbrook, J. H. Myers, H. Newman, N. Oppenheim.

BRENNEKE, AUGUSTUS MARTIN, Denison, Tex. (Age 38.) City Engr. of Denison. Refers to F. W. Chappell, O. J. S. Ellingson, H. E. Elrod, J. C. Feild, A. D. Stivers.

BURKE, JOSEPH EDWIN, Pittsburgh, Pa. (Age 41.) Gen. Mgr., J. K. Davison & Bro. Refers to H. A. Lane, P. G. Lang, Jr., R. H. Lee, R. Mather, H. F. Thomson, S. Walker.

CANTRAL, WAYNE WIOT, St. Paul, Nebr. (Age 23.) Refers to L. E. Conrad, M. W. Furr.

CARPENTER, EDWARD LARAMIE, Jr., Santa Monica, Cal. (Age 33.) With Puget Sound Bridge & Dredging Co., and W. F. Way. Refers to J. C. Greely H. M. Hadley, H. W. McCurdy, J. J. Manning, W. F. Way.

CARROLL, JOHN THOMAS, Glendale, N. Y. (Age 22.) Engr. Asst., Dept. of Public Works, New York City. Refers to A. Dick, B. Schwerin.

CHURCH, JAMES DUNCAN, Jr., Boulder City, Nev. (Age 25.) Asst. Engr., U. S. Bureau of Reclamation. Refers to J. J. Hammond, W. K. Hatt, J. L. Savage, H. W. Tabor.

CLAUS, THEODORE OTTO, Detroit, Mich. (Age 28.) Refers to A. J. Decker, W. C. Hoad, H. W. King, C. O. Wisler, J. S. Worley.

CONWELL, JAMES ALEXANDER, San Diego, Cal. (Age 22.) Refers to B. A. Etcheverry, G. E. Troxell.

DANIELS, FRANK EDWARD, Bridgeport, Conn. (Age 21.) Refers to E. R. Cary, L. W. Clark, H. B. Compton, T. R. Lawson, W. W. Rousseau, H. O. Sharp.

DAUDT, CARL FABIAN, Washington, D. C. (Age 25.) Refers to F. A. Biberstein, Jr., A. J. Scullen.

DE CERCE, FRANK, Syracuse, N. Y. (Age 23.) Refers to E. F. Berry, L. Mitchell, S. D. Sarason.

DERRY, STEPHEN JOHN-ARTHUR, Harrisburg, Pa. (Age 28.) Draftsman, Pennsylvania Dept. of Highways. Refers to H. K. Kistler, H. B. Shattuck, H. R. Thayer.

EBERLIN, WILLIAM GRAHAM, Perth Amboy, N. J. (Age 21.) Refers to E. R. Cary, W. W. Rousseau.

EDMONDSON, PAUL BAXTER, Amarillo, Tex. (Age 21.) Refers to L. B. Ryon, Jr., W. E. White.

EKLOFF, EDWIN EMANUEL, Croton-on-Hudson, N. Y. (Age 24.) Refers to F. E. Foss, M. H. Van Buren.

EMMANUEL, SAMUEL, Sind, India. (Age 28.) Supervisor, Public Works Dept. Refers to J. R. Colabawala, S. K. Gurtu. (Applies in accordance with Sec. 1, Art. I, of the By-Laws.)

ENGMAN, NORRIS VICTOR, Terre Haute, Ind. (Age 22.) Refers to R. E. Hutchins, R. L. McCormick.

ESTRIN, FRED, Kansas City, Mo. (Age 24.) With E. T. Archer & Co. Refers to E B. Black, E. Boyce, J. O. Jones, W. C. McNown.

EVANS, JACK CLARKSON, West Plains, Mo. (Age 25.) Refers to F. E. Fahy, B. M. Jones.

EVERNHAM, EARLE BARNETT, Neptune, N. J. (Age 25.) Refers to H. C. Bird, W. H. Hall.

FAY, ALBERT JAMES, Charleston, S. C. (Age 28.) Refers to E. R. Cary, L. W. Clark, T. R. Lawson, J. T. Mathews, H. O. Sharp.

FINK, HERBERT HENRY, Akron, Ohio. (Age 21.) Refers to F. E. Ayer, J. W. Bulger, R. C. Durst.

FLOYD, WESLEY RUFUS, Amesbury, Mass. (Age 22.) Rodman and Chainman, Univ. of New Hampshire, Durham, N. H. Refers to E. W. Bowler, G. W. Case.

FRY, JOSEPH ELMER, Jr., New York City. (Age 22.) Rodman, James Stewart & Co., Inc. Refers to A. M. Lund, W. R. Spencer.

GIBSON, OLEN CLAY, Jr., Norton, Va. (Age 21.) Refers to R. B. H. Begg, F. J. Sette.

GOSS, LOUIS HAROLD, Los Angeles, Cal. (Age 24.) Refers to R. R. Martel, W. W. Michael, F. Thomas.

GOULD, JACK HUMPHREY, Louisville, Ky. (Age 21.) Refers to J. A. Anderson, G. R. Smiley.

GUMENSKY, DMITRY BENJAMEN, Los Angeles, Cal. (Age 35.) With Metropolitan Water Dist. of Southern California. Refers to W. C. Christopher, C. C. Elder, J. Hinds, R. Matthew, F. H. Tibbetts, F. E. Weymouth, W. E. Whittier.

HANDLEY, DOCK HINTON, Jr., Teague, Tex. (Age 22.) Refers to L. E. Grinter, J. T. L. McNew, T. A. Munson, J. J. Richey, C. E. Sandstedt.

HARTMAN, PAUL, New York City. (Age 22.) Refers to B. A. Bakhmeteff, H. Crosby.

HEINRICH, CHARLES JOSEPH, Brooklyn, N. Y. (Age 26.) Refers to R. E. Goodwin, F. O. X. McLoughlin, J. C. Rathbun.

HILL, WILLIAM SHERMAN, Toledo, Ohio. (Age 25.) With City of Toledo, Ohio. Refers to C. M. Fyler, L. T. Owen, C. L. Piper, R. H. Randall, G. N. Schoonmaker, A. H. Smith, G. A. Taylor, G. D. Whitmore.

HOLLERAN, LESLIE GILBERT, Jr., Crestwood, N. Y. (Age 21.) Refers to R. A. Hall, A. G. Hayden, C. MacDonald, W. C. Taylor.

HOOK, HOWARD ARTHUR, Georgetown Station, Md. (Age 32.) Associate Civ. Engr., U. S. Dept. of Commerce, Div. of Airways, Washington, D. C. Refers to F. C. Hingsburg, J. R. Lapham, G. B. Skinner, G. W. Stephens, Jr., S. H. Stivers, G. E. Stratton.

HUNNELL, CHARLES, Jr., Vincennes, Ind. (Age 32.) Asst. Dist. Engr., Indiana State Highway Comm. Refers to G. H. Allen, D. J. Cloward, J. T. Hallett, R. E. Hutchins, R. L. McCormick, W. J. Titus.

IDEMA, JACOB, Beacon, N. Y. (Age 23.) Refers to T. R. Lawson, W. W. Rousseau.

IMPERIALE, MICHAEL ALOYSIUS, Brooklyn, N. Y. (Age 24.) Refers to H. R. Codwise, H. P. Hammond, L. F. Rader, E. J. Squire.

JAGOW, ARNOLD WILLIAM, Los Angeles, Cal. (Age 23.) Refers to E. L. Grant, J. B. Wells.

JERMANN, PAUL ANTHONY, Kenmore, N. Y. (Age 23.) Refers to L. M. Gram, L. C. Maugh, R. H. Sherlock, C. O. Wisler.

JOHNSON, CECIL WILLIAM, Seattle, Wash. (Age 26.) Refers to I. L. Collier, G. E. Hawthorn, C. C. More, R. G. Tyler.

JOHNSON, ROBERT KINNAIRD, Parkersburg, W. Va. (Age 35.) Agt., The Northwestern Mutual Life Insurance Co. Refers to H. L. Arbenz, T. F. Boltz, W. S. Downs, C. P. Fortney, W. O. McCluskey, Jr., H. McGraw, T. A. Polansky, H. E. Snyder, H. T. Sturm, P. J. Walsh.

KEMPTON, RAY ADDISON, Great Neck, N. Y. (Age 45.) Refers to T. C. Atwood, F. Auryansen, E. L. Collins, D. Y. Dimon, R. E. Dougherty, Z. H. Sikes, C. R. Weaver.

KESZLER, EDWARD ANTHONY, Philadelphia, Pa. (Age 23.) Refers to G. M. Fair, L. J. Johnson, H. M. Turner.

KOCH, ALBERT ARTHUR, Pasadena, Cal. (Age 23.) Refers to R. R. Martel, R. L. Stevens, F. Thomas.

KRACK, ROBERT CHARLES, New York City. (Age 21.) Laborer, Jiffy Join Co., Inc. Refers to T. R. Lawson, H. O. Sharp.

LAMANNO, EMANUEL SALVATORE, Bronx, N. Y. (Age 22.) Refers to J. J. Costa, A. V. Sheridan.

LAWTON, ELMORE GRENVILLE, Thacker, W. Va. (Age 20.) Refers to E. L. Clarke, D. D. Curtis, H. E. Glenn.

LAYNE, HENRY MARION, Whittier, Cal. (Age 26.) Refers to J. C. L. Fish, L. B. Reynolds, E. C. Thomas, J. B. Wells.

LIGGETT, PHILIP TAZWELL, Chicago, Ill. (Age 21.) Refers to J. O. Jones, W. C. McNown, F. A. Russell.

LOHMANN, ALEXANDER WILLIAM, East Elmhurst, N. Y. (Age 22.) Refers to E. R. Cary, L. W. Clark.

LUDEMAN, RICHARD HOUSTON, Brooklyn, N. Y. (Age 29.) Refers to A. H. Beyer, D. M. Burmister, J. K. Finch, W. J. Krefeld.

LYNCH, LEONARD THOMAS, Old Forge, Pa. (Age 27.) Refers to W. S. Lohr, L. Perry, E. H. Rockwell.

LYNGE, RAYMOND THOMAS, Urbana, Ill. (Age 26.) Refers to H. E. Babbitt, J. J. Doland, M. L. Enger, W. C. Huntington, T. C. Shedd.

MACDONALD, ROBERT GEORGE, Hollywood, Cal. (Age 23.) Refers to E. A. Burt, F. J. Converse, R. R. Martel, W. W. Michael, W. Moore, Jr., F. Thomas.

McKEAGUE, EDWARD DANIEL, Chicago, Ill. (Age 33.) Designer, Standard Oil Co. (Ind.), Whiting, Ind. Refers to R. A. Black, F. G. Brueckmann, H. Cross, W. G. Guild, W. C. Huntington.

MAESS, ALBERT CHARLES, Ballston Lake, N. Y. (Age 24.) Refers to R. A. Hall, H. Miller, W. C. Taylor.

MAIER, LOUIS HERMAN, Jr., New Rochelle, N. Y. (Age 23.) Refers to A. Haring, C. T. Schwarze, D. S. Trowbridge.

MARKS, SIDNEY MELVIN, Brooklyn, N. Y. (Age 24.) Refers to R. E. Goodwin, F. O. X. McLoughlin, J. Meltzer, G. Paaswell, J. C. Rathbun.

MEADOWCROFT, NIEL FAIRHURST, San Francisco, Cal. (Age 26.) Office Engr., Bridge Dept., Eng. Dept., U. S. Forest Service. Refers to E. L. Grant, C. G. Hyde, E. W. Kramer, J. H. Lawrence, D. R. McFarland, L. B. Reynolds.

MERCER, GLEN LUDLOW, Amarillo, Tex. (Age 36.) Asst. Engr., Western Lines, Atchison, Topeka & Santa Fe Ry. Co. Refers to M. C. Blanchard, I. C. Crawford, C. L. Eckel, A. E. Palen, J. W. Stewart, R. A. Van Ness.

METZGER, JOHN JOSEPH, Jr., New Iberia, La. (Age 22.) Engr., Jefferson Lake Oil Co., Inc. Refers to D. Derickson, A. T. Dusenbury.

MONNING, JOHN CHESTER, Jr., Los Angeles, Cal. (Age 21.) Chainman, Metropolitan Water Dist. of Southern California. Refers to E. A. Burt, R. R. Martel, F. Thomas.

MOORE, STEWART HUME, Hubbard Woods, Ill. (Age 23.) Refers to R. H. Sherlock, C. O. Wisler.

MOUSEL, FREDERICK WILLIAM, Mansfield, Ill. (Age 27.) Refers to T. C. Shedd, F. W. Stubbs, Jr.

MYERS, WILLIAM NELSON, Relay, Md. (Age 21.) Refers to J. H. Gregory, J. T. Thompson.

NEMMERS, ELMER JOHN, LeMars, Iowa. (Age 22.) Refers to R. B. Kittredge, B. J. Lambert, F. T. Mavis, F. A. Nagler.

NYE, HOWARD BOYD, Adams Center, N. Y. (Age 21.) Refers to L. Mitchell, S. D. Sarason.

PRICE, ROBERT LUMAN, Toledo, Ohio. (Age 20.) Refers to H. Bouchard, T. J. Mitchell, C. O. Wisler.

RAMIREZ, FRANK MILTON, Mayaguez, Puerto Rico. (Age 24.) Refers to G. H. Elbin, M. Font, F. Fortuno-Selles, E. Ortega-Rosado, R. Ramierz, R. Skerrett-Landron, A. R. Webb.

RASUL, MIR GHULAM, Cambridge, Mass. (Age 24.) Refers to G. M. Fair, A. Haertlein, L. J. Johnson, H. M. Turner.

RAUH, CHARLES EDWARD, Jr., Bogota, N. J. (Age 33.) With Western Elec. Co., New York City. Refers to M. T. Bolmer, T. L. Condron, V. McLean, C. L. Post, W. C. Rohdenburg, O. C. Spurling, T. K. Thomson.

ROBARTS, KENNETH WILLIAM, Oakland, Cal. (Age 46.) Office and Designing Engr., Alameda County Surveyor. Refers to C. Derleth, Jr., W. E. Emmett, A. S. Gelston, H. B. Hammill, B. Jameyson, R. F. Krafft, C. M. Mardel.

RUSSELL, FRANK McKLVEEN, Denver, Colo. (Age 25.) Refers to E. O. Bergman, R. L. Downing, C. L. Eckel, E. W. Raeder, A. W. Simonds, W. H. Thoman.

SANTORO, ALBERT PASQUAL, Brooklyn, N. Y. (Age 22.) Refers to H. R. Codwise, H. P. Hammond, L. F. Rader, E. J. Squire.

SILVERMAN, SAMUEL LELCHUK, Brooklyn, N. Y. (Age 21.) Refers to H. P. Hammond, L. F. Rader, E. J. Squire.

SMITH, EDWARD POWELL, Wilmington, Del. (Age 23.) Chf. of Survey Party, Delaware State Highway Dept. Refers to H. K. Preston, R. W. Thoroughgood.

SOLINS, DANIEL, Brooklyn, N. Y. (Age 21.) Refers to G. Morrison, J. P. J. Williams.

SPARLING, JACK NORMAN, Glendale, Cal. (Age 28.) Refers to F. J. Converse, R. R. Martel.

SPENCER, WILLIAM CLAY, Oakland, Cal. (Age 57.) Acting Chf. Bldg. Inspector, City of Oakland. Refers to A. H. Abel, O. E. Carr, W. G. Corlett, E. L. Driggs, W. N. Frickstad, H. Goodridge, H. F. Gray, J. B. Leonard.

STANLEY, JOHN WATSON, Empire, Colo. (Age 22.) Axeman, U. S. Bureau of Public Roads. Refers to F. R. Dungan, C. L. Eckel.

STEWART, JOHN A, Murray, Idaho. (Age 25.) Refers to F. E. Foss, J. H. Henderson, W. H. Yates.

STINE, WILLIAM VIRGIL, Litchfield, Ohio. (Age 23.) Refers to F. E. Ayer, J. W. Bulger, R. C. Durst

TAYLOR, ALVA ALMOND, Signal Hill, Cal. (Age 28.) Asst. City Engr., City of Signal Hill, Cal. Refers to A. H. Adams, R. E. Cranfield, F. S. Currie, R. F. Goudey, H. W. Jewell, E. M. Kelly, C. H. Shaw, J. S. Slover.

TEFFT, RICHARD COMSTOCK, New Berlin, N. Y. (Age 22.) Refers to R. A. Hall, A. deH. Hoadley, W. C. Taylor.

THOMAS, ALBERT COOKE, Baltimore, Md. (Age 20.) Refers to T. F. Hubbard, J. T. Thompson.

THORNTON, WILLIAM CLINTON, Bridgeport, Conn. (Age 22.) Refers to L. W. Clark, H. O. Sharp.

TIEDEMAN, FREDERIC WILLIAM, Brooklyn, N. Y. (Age 22.) Refers to H. P. Hammond, L. F. Rader, E. J. Squire.

TILLAPAUGH, HOWARD WESLEY, Torrington, Conn. (Age 23.) Refers to W. L. Foster, A. H. Fuller, L. O. Stewart.

VILLEPIQUE, ARTHUR GRANT, Glen Rock, N. J. (Age 21.) Refers to L. W. Clark, T. R. Lawson, H. O. Sharp.

WANDMACHER, FREDERICK CORNELIUS, Jr., Brooklyn, N. Y. (Age 22.) Refers to H. P. Hammond, L. F. Rader, E. J. Squire.

WEINBERGER, SAMUEL AARON, Bronx, N. Y. (Age 24.) Refers to R. E. Goodwin, F. O. X. McLoughlin, J. C. Rathbun.

WHITEHEAD, ROBERT LEE, Wake Forest, N. C. (Age 42.) Res. Engr., North Carolina State Highway Comm. Refers to M. F. Larson, T. Pettigrew, G. F. Syme, D. Ulrich, J. D. Waldrop.

WICKSTROM, WICK, New York City. (Age 35.) Chf. Engr., Warren & Arthur Smadbeck, Inc. Refers to J. N. Brooks, H. T. Critchlow, J. P. Newton, C. H. Sells, P. L. Voss.

WILLSON, ELMER JAMES, Sherman, Tex. (Age 21.) Refers to J. T. L. McNew, J. J. Richey.

WITZIGMAN, FREDERICK SCOTT, Iowa City, Iowa. (Age 27.) Surveyman, U. S. War Dept., Corps of Engrs. Refers to E. E. Moots, F. A. Nagler, C. C. Williams, S. M. Woodward, D. L. Yarnell.

WONG, CHEO-CHUN, Ann Arbor, Mich. (Age 27.) Refers to W. J. Emmons, L. M. Gram, R. L. Morrison, W. C. Sadler, R. S. Swinton, J. S. Worley.

WOOD, ROBERT HART, Jackson, Miss. (Age 22.) Structural Draftsman and Estimator, Bridge Div., Mississippi State Highway Dept. Refers to S. C. Hollister, D. M. McCain.

WOODS, GEORGE ALAN, Bronx, N. Y. (Age 29.) Refers to R. E. Goodwin, F. O. X. McLoughlin.

WYMAN, JACK A, Omaha, Nebr. (Age 31.) Refers to R. E. Edgecomb, M. L. Enger, N. D. Morgan.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

CONE, RUSSELL GLENN, Assoc. M., San Francisco, Cal. (Elected Junior Dec. 4, 1922; Assoc. M. Jan. 18, 1826.) (Age 37.) Res. Engr., Golden Gate Bridge and Highway Dist., for Strauss Eng. Corporation. Refers to C. E. Chase, H. Cross, M. S. Ketchum, F. M. Masters, R. Modjeski, C. E. Paine, H. J. Sherman, G. B. Woodruff.

CONLON, WILLARD SYLVESTER, Assoc. M., Stamford, Conn. (Elected May 25, 1931.) (Age 38.) City Engr. and Mgr. of Public Works Dept. Refers to H. R. Buck, W. W. Drummond, G. P. Morrill, A. R. Norcross, H. B. Perry, E. P. Quirk, W. J. Scott, A. V. Sheridan, D. B. Steinman.

DOCKSTADER, ERNEST AMBROSE, Assoc. M. Boston, Mass. (Elected April 3, 1922.) (Age 43.) Senior Structural Engr., Stone & Webster Eng. Corporation. Refers to F. O. Dufour, A. L. Hartridge, F. E. Hertel, W. N. Patten, G. R. Strandberg, L. J. Towne.

EDWARDS, RAY OMER, Assoc. M., Clearwater, Fla. (Elected Oct. 14, 1930.) (Age 41.) Field Engr., Portland Cement Association, Atlanta, Ga. Refers to A. Blair, W. K. Hatt, H. D. Mendenhall, E. R. Neff, W. I. Nolen, R. B. Wiley.

HOU, CHIA YUEN, Assoc. M., Nanking, China. (Elected Junior March 7, 1921; Assoc. M. Aug. 27, 1928.) (Age 36.) Director of Public Works, Nanking (China) Municipality. Refers to F. A. Barnes, D. S. Gendell, Jr., H. S. Jacoby, R. MacMinn, A. M. Shaw, G. L. Taylor.

POPE, LATHROP CARLETON, Assoc. M., Flushing, N. Y. (Elected Jan. 19, 1925.) (Age 41.) Res. Engr., The Carleton Co., Inc., Brooklyn, N. Y. Refers to J. H. Cissel, L. M. Gram, J. H. C. Gregg, A. Lyle, N. D. Richardson, A. M. Wyman.

ROACH, JAMES HOWARD, Assoc. M., New York City. (Elected April 5, 1910.) (Age 51.) Chf. Valuation Engr., New York Central Lines. Refers to E. Y. Allen, W. G. Atwood, M. C. Cleveland, H. E. Hale, A. P. Hoover, G. W. Kittredge, B. R. Leffler, F. B. Scheetz, C. Silliman, E. F. Wendt.

RUSSELL, LEWIS WALTER, Assoc. M., Chicago, Ill. (Elected March 14, 1927.) (Age 40.) Engr. and Supt., Bates & Rogers Constr. Co. Refers to W. G. Arn, C. N. Bainbridge, F. L. Copeland, M. Goodkind, W. A. Rogers, H. O. Schermerhorn, G. H. Tinker.

SHIELDS, JAMES RALPH, Assoc. M., Sacramento, Cal. (Elected Junior Oct. 1, 1912; Assoc. M. March 12, 1917.) (Age 48.) Bldg. Inspector, City of Sacramento, Cal. Refers to D. Butler, H. D. Dewell, A. Givan, J. W. Gross, H. N. Jenks, C. H. Kromer, M. C. Poulsen.

FROM THE GRADE OF JUNIOR

BAKER, PAUL WALLACE, Jun., Omaha, Nebr. (Elected July 15, 1929.) (Age 27.) Asst. Constr. Engr., Metropolitan Utilities Dist. of Omaha. Refers to W. H. Campen, M. I. Evinger, W. L. Fulton, T. A. Leisen, B. Manchester, C. E. Mickey, R. N. Towl.

BRACKETT, WOODBURY EDMUND, Jun., Washburn, Maine. (Elected Aug. 17, 1931.) (Age 30.) Town Mgr., Town of Washburn. Refers to W. F. Day, W. T. Lyle, W. J. Mathews, Jr., R. H. Patterson, E. S. Thomas.

COLTHARP, JOHN ROBERT, Jun., Austin, Tex. (Elected Dec. 28, 1931.) (Age 27.) Chf. Engr., Southwestern Aerial Surveys, Inc. Refers to E. C. H. Bantel, O. G. Baxter, C. H. Birdseye, G. R. Clemens, A. H. Dunlap, E. Haquinius, J. A. Norris, T. U. Taylor, W. O. Washington.

COMBS, THEODORE CARLOS, Jun., Upland, Cal. (Elected Oct. 14, 1930.) (Age 27.) Bldg. Inspector, City of Upland. Refers to R. G. Manley, R. R. Martel, D. H. Merrill, F. Thomas, C. D. Wailes, Jr.

EVERHART, CALVIN CECIL, Jun., Moville, Iowa. (Elected Nov. 12, 1928.) (Age 32.) In private practice. Refers to P. D. Cook, J. H. Griffith, S. Klein, D. E. Moran, E. A. Prentis, R. S. Saunders, E. B. Wilson.

REANTILLO, TEOFILO PASCUA, Jun., Jersey City, N. J. (Elected Nov. 23, 1931.) (Age 30.) Computer, U. S. Coast and Geodetic Survey, New York City. Refers to J. C. Engelhart, T. Human, Jr., S. Johannesson, H. A. Kelly, H. H. Pitcairn, L. R. Shellenberger, E. J. Squire.

SOUCEK, JAROMIR JAN, Jun., Prague, Czechoslovakia. (Elected Nov. 11, 1929.) (Age 32.) Cons. Engr. in Prague. Refers to C. B. Burdick, L. R. Howson, D. H. Maxwell, L. M. Mensch. (Applies in accordance with Sec. 1, Art. I of the By-Laws.)

THOMPSON, WALTER EDWIN, Jun. Brooklyn, N. Y. (Elected Jan. 16, 1928.) (Age 32.) Designing Draftsman, New York Zoological Park, Bronx, N. Y. Refers to C. F. Blood, D. W. Krellwitz, C. J. Renner, T. M. Ripley, W. L. Selmer, D. B. Steinman, J. F. Williamson.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.